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Q Street Bridge at Washington

The valley of Rock Creek, separating from the main part of the city that section of Washington, D. C., which was at one time the old city of Georgetown, is a deep winding gorge through which a small stream flows to the Potomac. The increasing growth of the city toward the north and the west has made necessary within the past ten to fifteen years the construction of a number of high-level bridges across the gorge. Four of these bridges are of the arch type—the old Massachusetts Ave. viaduct, a brick, small-opening arch with a heavy fill, more like a culvert than a bridge; the Connecticut Ave. bridge, finished about ten years ago, well known as one of the most beautiful bridges in the country; the Pennsylvania Ave. bridge, which incloses the old water-main arch of the Washington aqueduct and which was described in *Engineering News* of Dec. 16, 1915; and finally the Q St. bridge, which is the subject of this article.

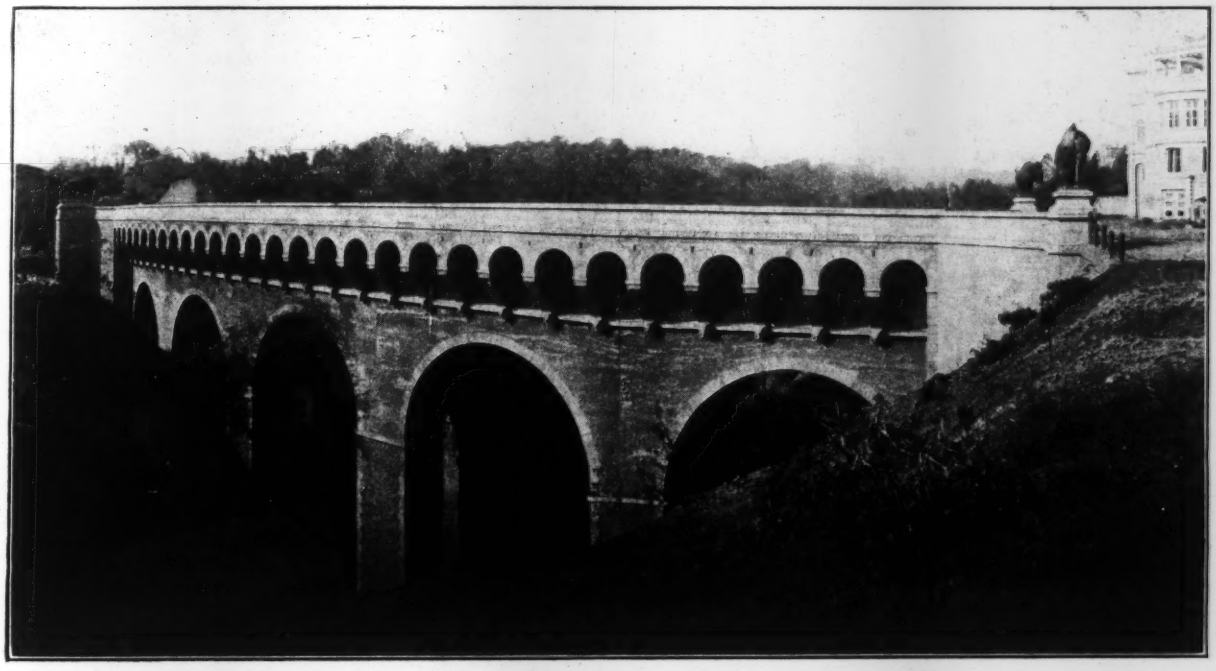


FIG. 1. THE NEW Q ST. BRIDGE ACROSS ROCK CREEK, WASHINGTON, D. C.

The Q St. bridge is noteworthy because of the special effort that was made in its design to have it conform architecturally with the very high-class residences that are going up around the east end of the bridge, and to fit in with the proposed park system that is expected soon to reach down the valley from the famous Rock Creek Park beyond. Unfortunately at present the west end of the bridge and the valley itself are in a decidedly unkempt condition, not at all in keeping with the other end of the bridge or the bridge itself. However, this state of affairs will probably soon be changed.

In the desire to achieve a desirable architectural effect the city officials called in a consulting architect in the person of Glenn Brown, who was formerly secretary of the American Institute of Architects, and it is to Mr. Brown and to the higher officials in the District government rather than to the city engineers that the decorative features of the bridge are due. These effects are evident not only in the decoration, adornment and color of the bridge, but also in the fact that it is built on a curve in plan, although there seems to be no very good structural reason why this extremely difficult and annoying type of construction should have been adopted. It is understood, however, that someone in higher authority had been particularly impressed with the artistic effect of the famous old curved arch viaduct of the Baltimore & Ohio R.R. at Relay, Md., just outside of Baltimore, and took occasion to use the same general scheme for the Q St. bridge. The plan is partly allowable because the approach streets on the opposite sides of the gorge are not in alignment.

The bridge for the time being will be used only as a highway crossing. It consists of five arch spans varying

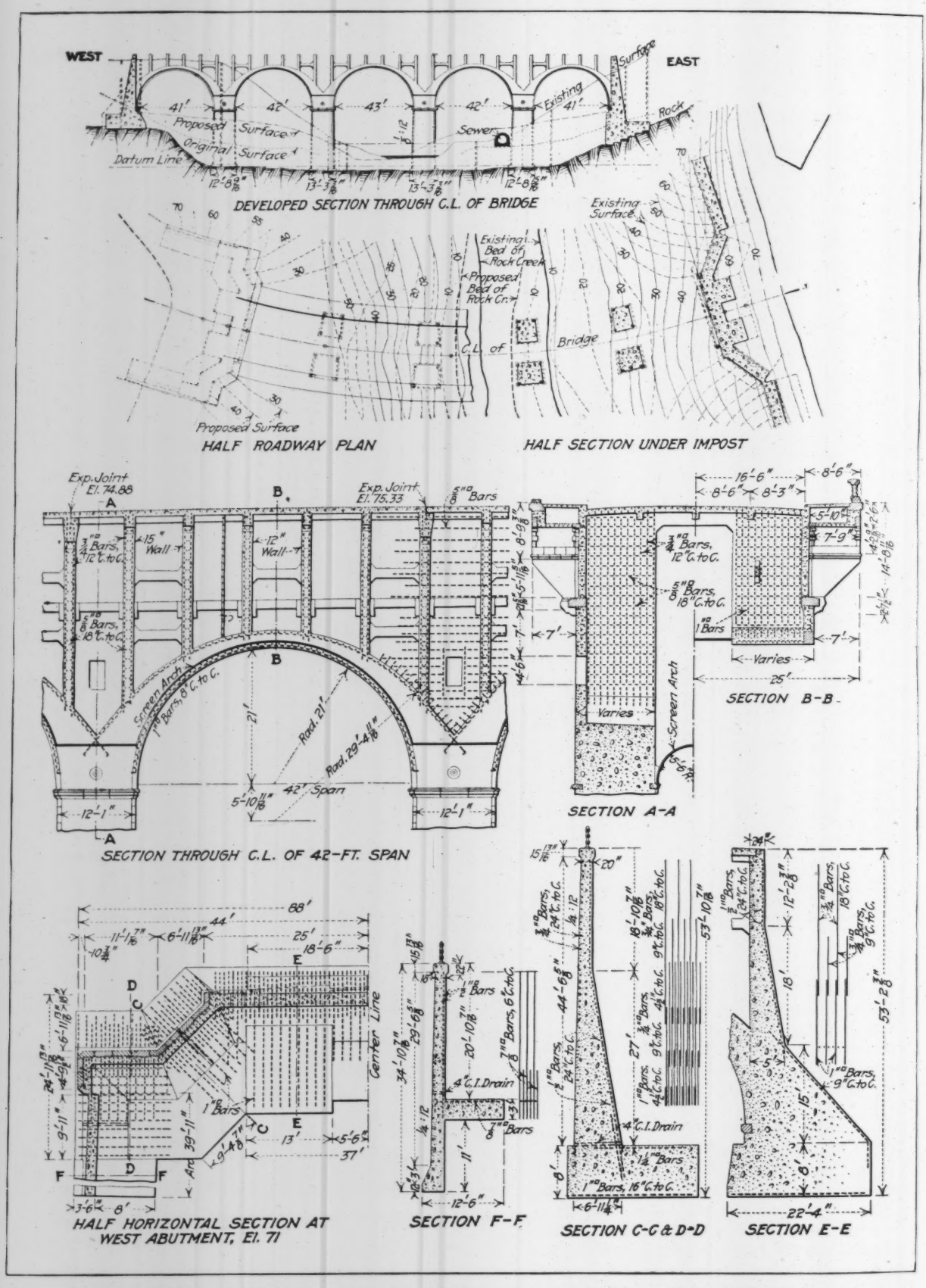


FIG. 2. DETAILS OF THE NEW Q ST. BRIDGE ACROSS ROCK CREEK GORGE, WASHINGTON, D. C.

from 43 ft. clear span in the center to 42 ft. for the next adjoining two, and 41 ft. for the end spans. It carries two 7-ft. sidewalks and a 33-ft. roadway. There are twin arch ribs, each of solid reinforced section footing into separated piers. Screen arches of thin section span between the main arches to form a solid soffit.

The reinforced-concrete floor slabs are carried on two-way beams that rest on transverse walls reaching down to the arch ribs. These walls are cross-braced longitudinally, as shown in Fig. 2. Solid spandrel walls form a showing face, as may be noted from the view in Fig. 1. Sidewalks are carried on overhanging beams that are joined with arches in a highly ornamental fashion, as noted in the view of the whole arch.

The center line of the roadway is curved to a radius of 474 ft. The piers are curved on their outer face and dimensioned to fit the radii, as shown in the plan Fig. 2. The arch ribs themselves, as well as the spandrel walls, are also bent to this same curvature. In construction this did not cause as much difficulty as might be imagined, because the arches are so short that the offset was in no case very large. The curve is carried to the roadway and shows in the curbstone and in the railing. It is so slight that a casual observer on the bank, so far away as not to be able to see the roadway with its curbs and railing, might not realize that the bridge is curved in plan unless the eye took in the line of the street continuing the bridge on the far side. In the first-page view this street may be noted as the white strip extending to the right at the far side of the bridge.

Details of Abutment Design

The abutments are of the wing type and somewhat unusual in design. Fig. 2 shows some of the details of the west abutment. The other is quite similar. Both abutments and the piers foot on rock. As will be noted in the drawing, the wings are of complex plan, but the general type of construction is a thin retaining wall backed with earth. At the arch rib the wall is extended backward to take care of the thrust from the arch, but the middle section, in Fig. 2, is typical of the other parts of the wall. This wall is 46 ft. high to the top of the solid 8-ft. concrete base. It is 1 ft. 8 in. thick down to 18 ft. below the top and increases uniformly in thickness from that point to the top of the base wall, where it has a thickness of 7 ft., reinforced as shown in the section.

At the rear of the wing wall, where the slope of the bank began to come up so high on the wall that it was not thought worth while to excavate clear to the bottom, the wall itself was carried down to a depth required by the showing face. The fill was made on top of a relieving wall 3 ft. thick, which rested on the existing earth and which was joined by steel and concrete to form an integral part of the main wall. In this way the thrust on the showing wall was considerably reduced. The faces of all of the walls are slightly battered.

It might be noted that in view of a disastrous experience some ten years ago in the Connecticut Ave. bridge in Washington, the engineers of the city made special provision that the abutment between the outer lines of the bridge should not be completely back-filled until after the arch centers were struck.

The decorations and ornaments to the bridge are a special feature. The tone of the bridge is red, the main part of the spandrel walls being of a reddish concrete bush-hammered for rough surface. The quoins, coping, railing and abutment faces are of a red sandstone. The specifications required that the concrete face should be made of an aggregate formed from the crushed sandstone which went into the cut stone of the bridge, but when a test concrete was made in this way it was found that the cement and sand so nullified the reddish tint of the crushed sandstone as to give a concrete in no way resembling in tone the original sandstone. After considerable experiment and search, however, a red sand was found in a pit in Virginia near Washington which, when mixed with cement, formed a mortar very similar in tone and color to the sandstone. Unfortunately the effect of the bushhammered concrete is somewhat lessened by

FIG. 3. ONE OF THE BRONZE BUFFALOES ON THE Q ST. BRIDGE
A. Phimister Proctor, sculptor

streaks in the concrete face where patches of different-colored concrete got against the form. This may be noticed in the view in the first page. It may disappear in time when the concrete surface has weathered somewhat.

In addition to the ordinary bridge decorations in the form of cut-stone quoins and coping, the Q St. bridge has also very ornamental carving of the railing and the carved "Aztec's head" at the base of each cantilever. In addition the center of each spandrel wall between arches has a highly ornamental monogram D. C., for the District of Columbia. Another innovation is the placing of the lamp posts on "isles of safety" on the center line of the roadway.

The crowning decorations are the four huge bronze buffaloes which face each other on the pedestals at each approach. These buffaloes were modeled by A. Phimister Proctor, of New York, and cost the City of Washington about $10,000. They make fitting companion animals to the lionesses of the Piney Branch bridge and the lions of the Connecticut Ave. bridge of the same city. A view of one of these bronzes is shown in Fig. 3.

The Q St. bridge was built by A. L. Guidone & Co., of New York City, under the general direction of D. E. McComb, Engineer of Bridges of the District of Columbia. Glenn Brown, of Washington, D. C., as stated above, was consulting architect.

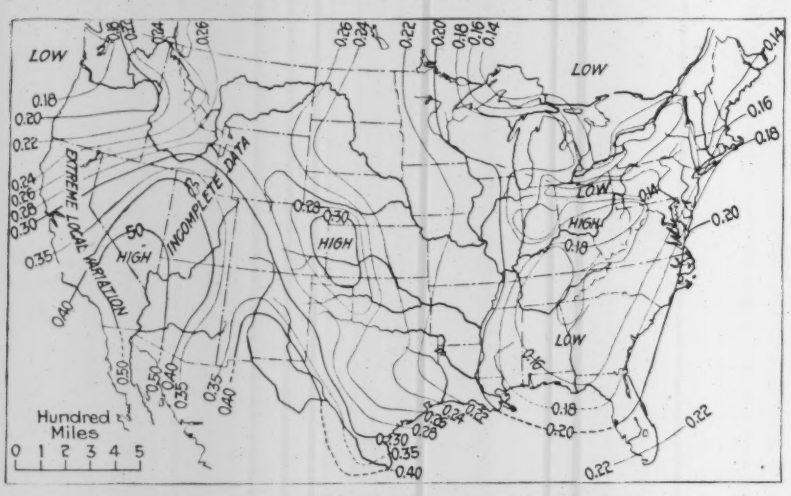
Variation in Annual Rainfall

By Allen Hazen*

Good maps showing the normal annual precipitation throughout the United States are published by the Weather Bureau and by the United States Geological Survey. Numerous maps of states show normal annual rainfall with more precision, because a smaller interval between the lines can be used with the larger scale. With the aid of these maps and the full records of the Weather Bureau, which can be obtained from the Government, it is possible to get quickly a fair idea of the mean annual rainfall conditions at any point in the United States, as far as shown by existing data.

The variation in rainfall is a matter of almost as much practical importance as its amount. The seasonal variations may be found from certain Weather Bureau tables that show the normal precipitations by months, and also from certain maps showing seasonal fluctuations. The annual variations are equally important, but are not as easily ascertained. In hydraulic literature their treatment has been inadequate.

One of the most noteworthy investigations of this subject is that of Sir Alexander Binnie, presented many years ago to the Institution of Civil Engineers[1] and more recently without essential change in his book on water-supply[2].



COEFFICIENT OF VARIATION OF ANNUAL RAINFALL IN THE UNITED STATES

During the past year the writer has made an effort to ascertain what may be expected in the way of variation in annual rainfall in the United States. This work has been done to aid in understanding rainfall and runoff data.

Measures of Variation

There are many possible measures of variation. The one selected as most suitable for this study is the "coefficient of variation" as defined in works on statistics. It is easily obtained by calculation from the official records and seems better adapted to the service than any other that has been suggested. The coefficient of variation is determined by the following simple procedure:

The annual rainfalls for each year of the whole record period are obtained. The mean of this series is usually published by the Weather Bureau at the bottom of the column. The difference between the actual rainfall for any year and the mean of the whole record period is the variation (from the mean) for that year. These variations are found for each year of the series. Each one is then squared. The mean of these squares is then found, and its square root is taken. This is the standard variation. The coefficient of variation is the ratio of the standard variation to the mean. A few short-cuts are possible, and after a little practice the coefficient can be found from a 40-yr. record in about 40 min.

Published records of rainfall were obtained from the Weather Bureau. They frequently covered the records only up to 1908 or 1909. The more recent records are available in other publications, but to use them involved new listing and computation of the means. For this purpose the additional listing and averaging needed to make them available were not undertaken. Not all the records were used. Many of them were too short to be of much service. In general the longest and apparently best records, scattered as evenly as possible all over the country, were selected to the number of several hundred. The coefficients for each series were determined and plotted on an outline map of the United States. On the basis of this information, lines were drawn to show the normal coefficients of variation all over the country.

When the points first calculated and plotted seemed an inadequate basis for drawing the normal lines on the map, the Weather Bureau reports were again examined and additional records were selected to help fill in. Although shorter, and having been for that reason rejected at the time the first selection was made, these were used, until finally coefficients calculated from over 500 records had been plotted and used in defining the positions of the normal coefficients of variation. A number of Canadian records were used to aid in drawing the lines near the border.

On the whole the information was consistent. Only a very few stations were found giving coefficients so far from their neighbors as to suggest some unusual condition or error. A very few such records were excluded.

For the eastern part of the United States the records used are sufficient to show the normal conditions with a fair degree of accuracy. In the West the relative variations are greater, the records are shorter and the stations often farther apart and the results thus less reliable.

The map made up in this way is reproduced herewith. This is to be regarded as a first rough approximation. It serves to give a fairly accurate idea of the general conditions of variation in annual rainfall in the United States, but in detail it must be expected that more ample data, or even more complete use of existing data by extending the studies to cover everything available, would result in changes in the positions of the lines. Such changes

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[1]Minutes of Proceedings of the Institution of Civil Engineers, Vol. XXXIX, Session 1874-5, Part I, and CIX, Session 1891-92, Part III.

[2]"Rainfall, Reservoir and Water-Supply," page 12.

are most likely near mountains and in about that third of the country nearest the Pacific Coast.

The coefficient of variation is lowest on the Atlantic Coast. That means that one can count on getting more nearly the normal rainfall each year; and on the Atlantic Coast the coefficient does not range through wide limits. If one's experience were limited to this part of the country, he might almost neglect the range and assume that there was everywhere the same chance of a very dry year, such for example as is represented by a rainfall of only 60% of the normal. This is in general the conclusion reached by Binnie for the data studied by him. It does not hold, however, for the whole United States. As one goes west, the coefficient of variation in annual rainfall increases; that is to say, the chance of a very dry or a very wet year (relatively) is increased.

At New York the coefficient of variation is 0.15; at San Francisco it is 0.30. This means that on an average the relative variations in annual rainfall at San Francisco are twice as great as at New York. A year 40% short of the normal rainfall comes as often at San Francisco as one 20% short of the normal comes in New York.

By following this method of investigation it will not be hard to find quite definitely just how often, on an average, a year of any assumed relative wetness or dryness will come, and also to find what the chances are of years of any degree of dryness coming within a given period.

If the rainfall records followed exactly the "normal" law of error, a very simple procedure could be outlined that would lead directly to the result; but the law that governs the rainfall is just a little different from the "normal" law of error, and the difference is enough so that it is not best to attempt to make the calculation by its use. At present a graphical solution using probability paper (see Am. Soc. C. E., Vol. LXXVII, p. 1539) seems to be the surest way of handling the matter, and the safest way to get results applicable to a particular locality is to use the local data.

Other things being equal the variations to be expected in any period at different localities will always be in direct proportion to the coefficients of variation in the respective places.

The work of making the computation and of drawing the map was mainly done by R. H. Eurich.

Solving the Problem of Local Engineering Associations

What seems to be an excellent solution of some of the problems noted in the editorial, "Coöperation or Amalgamation of Local Engineering Societies," in *Engineering News*, Nov. 25, 1915, is contained in the new regulations of the Associated Engineering Societies of St. Louis, which is composed of the Engineers' Club of St. Louis, the St. Louis Association of Members of the American Society of Civil Engineers, and the St. Louis branches of the American Society of Mechanical Engineers, American Institute of Electric Engineers and the American Society of Engineering Contractors. Other local organizations of 50 or more members which are branches of national engineering societies are eligible for affiliation.

The affairs of the association are conducted by a joint council consisting of the president and two junior past-presidents of the Engineers' Club, together with two other councilors chosen by each of the affiliated societies, one each year for a term of two years, at least one of whom is to be a member of the affiliated society's governing committee, and at least one a member of the Engineers' Club. The president, treasurer and secretary of the Engineers' Club act in similar capacities for the council. The council has jurisdiction over all matters of joint interest.

Joint Action on Local Affairs

The societies of the association act jointly on all local public affairs presented for consideration to any one or more of the societies. Such questions are referred to the council for its recommendation, and upon a majority vote they are put to a referendum vote of the entire membership. In national affairs the recommendations of the council, if any, are printed, and if they obtain a three-fifths vote in favor, are referred to the consideration of one or more of the national engineering bodies.

Each society is to arrange per calendar year two technical joint meetings in which all the other societies participate. One joint banquet annually and such joint meetings as may be arranged by the council are also held. Each affiliated society arranges all technical meetings other than joint meetings as "party meetings" in which only the affiliated society and the members of the Engineers' Club participate. Each individual affiliated society, however, may arrange nontechnical "individual" meetings in which it alone may participate.

The auditorium of the Engineers' Club is available for all joint and party meetings, and the members of the Engineers' Club have the privilege of participating in the discussion of any subject under discussion. In recognition of the Engineers' Club furnishing the auditorium, library, clubroom, etc., it is recognized that the St. Louis Engineers' Club is the parent society and reserves the right to hold technical and society meetings and excursions with or without the participation of one or more of the affiliated societies.

Equitable Distribution of Costs

Dues are based on the estimated cost of the meetings: $10 for rent of the auditorium; $10 for secretary service; cost of printed notices, etc. Statements of each society's accounts are rendered quarterly by the secretary of the Engineers' Club and each society, including the Engineers' Club, pays its pro rata share of the cost of meetings, based on the total number of members on its mailing list, and in the case of affiliated societies the number of members on its mailing list who are not members of the Engineers' Club. For individual meetings a rental charge of $10 for the auditorium is made.

In consideration of the Engineers' Club offer to share its clubroom, it is agreed that each affiliated society shall encourage its members to join the club and that the Engineers' Club will encourage its members to join the national engineering societies.

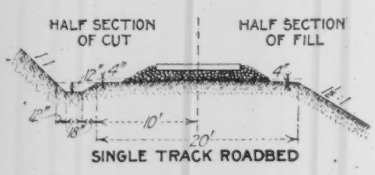
The papers of the various organizations may or may not be published in the monthly journal of the Engineers' Club; but there is to be published each year a Year Book of the associated societies, for which each society pays in proportion to its membership.

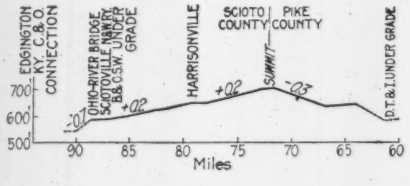
Any affiliated society which fails in its obligation after being notified by the council may at the end of three months be dropped from the association by a majority vote of the council. Any associated society may withdraw from the association after three months' notice.

Building the Chesapeake & Ohio Northern Railway

SYNOPSIS—Total excavation 3,600,000 yd., of which 25% is rock. Cuts and fills 90 ft. deep. Special methods of cutting. Spreaders used on rock fills. Hauls up to 5½ mi. Gumbo caused slides in cuts and proved unstable in fill; how trouble was cured. Difficult movement of heavy plant.

Heavy rockwork is met in the construction of the single-track Chesapeake & Ohio Northern Ry. between Sciotoville and Waverly, Ohio, on the first 7 mi. north of the Ohio River. Over the 28 mi. of new line four general contractors and about a dozen subcontractors are cutting and filling, and erecting culverts and a viaduct. As originally proposed the railway was to run from Columbus to Edgington, Ky., where it was to connect with the Chesapeake & Ohio Ry. The line was located between these two points; but owing to financial conditions, at the present time only the 38-mi. strip is being constructed between Limeville (2 mi. west of Edgington) and Waverly, at which point the line has track connection with the Norfolk & Western Ry., which continues to Columbus and there connects with the track of the Hocking Valley Ry. (extending to Toledo). Thus by constructing the Chesapeake & Ohio Northern, the Chesapeake & Ohio Ry. obtains a desirable outlet to the Great Lakes.





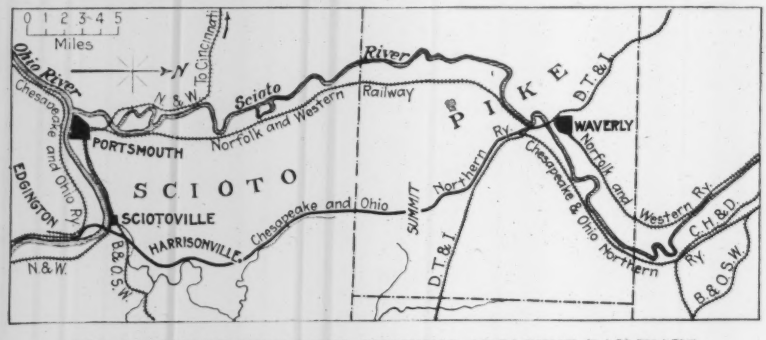


FIG. 1. MAP OF NEW LINE, SHOWING ADJACENT RAILWAYS

The maximum grade and curvature of the new line is 0.2% compensated against northbound traffic and 0.3% against southbound, with the exception of a northbound pusher grade of 0.7% on the Kentucky approach to the Ohio River bridge. Sidings 6,000 ft. in length are provided every 5 mi. They are flattened out to facilitate starting heavy trains from standstill on the siding.

Contractors—Work They Must Do

The building of the new line is supervised by the railway under three residences and is divided into four contract sections. Starting at the Kentucky end the general contractors are: Section 1 (3 mi.) W. W. Boxley, of Roanoke, Va., having 450,000 yd. of earthwork and 1,500 yd. of reinforced concrete; Section 2 (7 mi.), Rinehart & Dennis, of Charlotteville, Va., having 700,000 yd. of rockwork, 600,000 yd. of earth and 25,000 yd. of concrete; Section 3 (9 mi.), Winston & Co., having 80,000 yd. of rock, 800,000 yd. of earth and 10,000 yd. of concrete; Section 4 (9 mi.), Robert Grace Contracting Co., of Pittsburgh, Penn., having 120,000 yd. of rock, 850,000 yd. of earth and 15,000 yd. of concrete. The Dravo Contracting Co., of Pittsburgh, built the substructure of the Ohio River bridge; the McClintic-Marshall Construction Co. will erect the superstructure. A considerable part of the work is sublet.

The total excavation is estimated at 900,000 yd. of rock and 2,700,000 yd. of earth, Section 2 having 80% of the rock. Included in the construction are: The Sciotoville two-span continuous-truss bridge over the Ohio River, 1,550 ft., with viaduct approaches of 700 ft. on the Ohio side and 1,100 ft. on the Kentucky side (foundations completed having 27,000 yd. of concrete); a 1,080-ft. viaduct over the Little Scioto River; an overhead crossing of the Baltimore & Ohio Southwestern Ry.; 12 highway undercrossings; 6 overhead crossings; 3 grade crossings; numerous concrete culverts; 2 timber trestles on the temporary connection with the Norfolk & Western and a number of strips of county and township road changes. Concrete work amounts to 86,000 cu.yd., of which 32,000 yd. is reinforced; 14,500 tons of steel, including 12,200 for the Ohio River bridge is required. The first shovels started work in May, the preliminary work having been completed that same month. Work will continue through the winter, and it is not expected to be completed for at least a year.

How the Work Is Being Done

On the Kentucky side of the Ohio River a 52-ft. fill containing 450,000 yd. of borrow-pit earth forms the southern approach to the Sciotoville bridge. Record work is being accomplished at the borrow pit by a Marion 61 (2½-yd. dipper); an average of 43,000 to 44,000 yd. is being excavated per month. The biggest month for this shovel, with one 11-hr. shift working 6 days per week, was 53,000 yd. The fill is transported from the pit in 6-car or 7-car trains of 12-yd. Western air-dump cars drawn by 60-ton locomotives on a standard-gage track. A siding and a water tank are provided at the pit. The haul is 1½ mi., and two trains transport the material from the pit to the fill.

The filling trestle is 23 ft. high, having the usual two-pole bents spaced 6 ft. apart. It is capped with ties, and between bents an extra tie is spiked to the rail. By the middle of November the trestle had been filled to the December. The subcontractor on this work is C. Y. Lygon & Co., of Prestonburg, Ky.

Meanwhile a temporary trestle had been thrown over the Baltimore & Ohio Southwestern Ry., so that rock

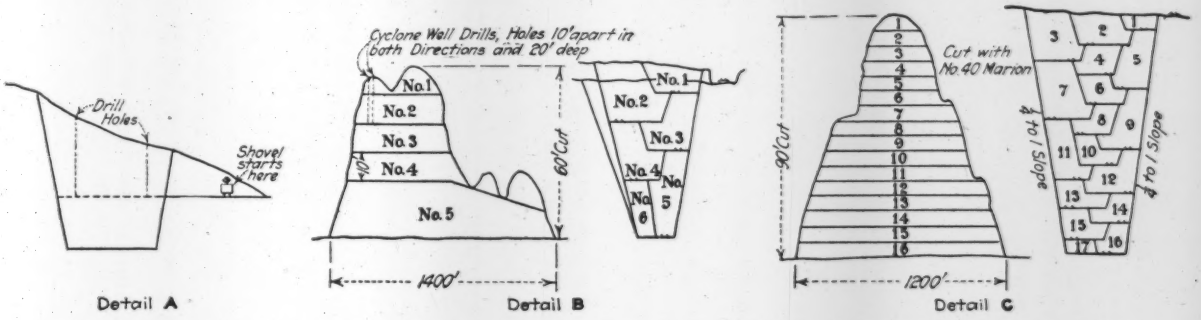


FIG. 2. METHOD OF EXCAVATING DEEP ROCK CUTS ON THE CHESAPEAKE & OHIO NORTHERN RY. WORK

top, and the track is now being jacked by track jacks as the fill rises. A western spreader is used.

Just north of the Ohio bank an 88,000-yd. semi-sidehill cut 55 ft. deep in rock is being taken out. A cross-section of the cut is shown in Fig. 2. As indicated, although really a through cut, it is being worked as a sidehill cut.

Drill holes were put down from the top by well drills. The holes were spaced about 25 ft. apart. They were first sprung with three springing charges of 15, 100 and 200 lb. of dynamite respectively; the blasting charge averaged 2,000 lb. This was tamped with clay by long wooden poles. The heaviest blast consisted of 11,000 lb. of dynamite placed in five 65-ft. holes. The rock is Waverly shale.

Clearing rock from this sidehill cut and from two more near-by (225,000 cu.yd. together) are two shovels—an 80-C Bucyrus (3½-yd. dipper) and a Marion 61. It is remarkable what large pieces can be manipulated by the shovels. The heaviness of the work is attested by the big rocks on the adjoining fill. Rocks of 14 cu.yd. have been worked around until the dipper gets a purchase on them, when they are lifted—only a point being in the shovel—and deposited in the cars.

Kilbourne & Jacobs air-dump cars of 18 yd. each transport the rock to the fills, and there a Jordon spreader, supplied by air from the locomotive, levels it. To clear the cars of large rock, it is necessary to remove one side of the car. This is a time-consuming task, as the cars are very heavy, but accidents have proved the necessity for this precaution. Two of these cars rolled down a 200-ft. bank into the Little Scioto River on one occasion when this practice was not adhered to. The wrecked cars had been insured for $1,000 each. The insurance was collected, and the cars will be rebuilt at a later date by the contractor. The material trains run on a standard-gage track. For long hauls seven or eight cars make up a train; on short hauls or steep grades three or four cars are sufficient. Langhorne, Langhorne Co. & Sneed are working these three cuts and fills.

The line crosses over the Baltimore & Ohio Southwestern Ry. a half mile south of the Little Scioto River on a 26° skew. The main track and a siding are carried over on 107-ft. deck girders 35 ft. above the Baltimore & Ohio Southwestern track. The abutments are carried on piles driven into blue clay. They were completed in

could be carried across to the fill, which extends from the crossing to the Little Scioto viaduct. This fill contains 220,000 yd., of which 50,000 is borrowed earth and the rest is rock.

The viaduct over the Little Scioto River is 1,080 ft. long. Its concrete pedestals and abutments have been completed. Upon them steel towers will be erected. The first (south) four pedestals and the south abutment bear on compact clay; the remainder are either directly on rock or are on piles driven to rock. The contractor on

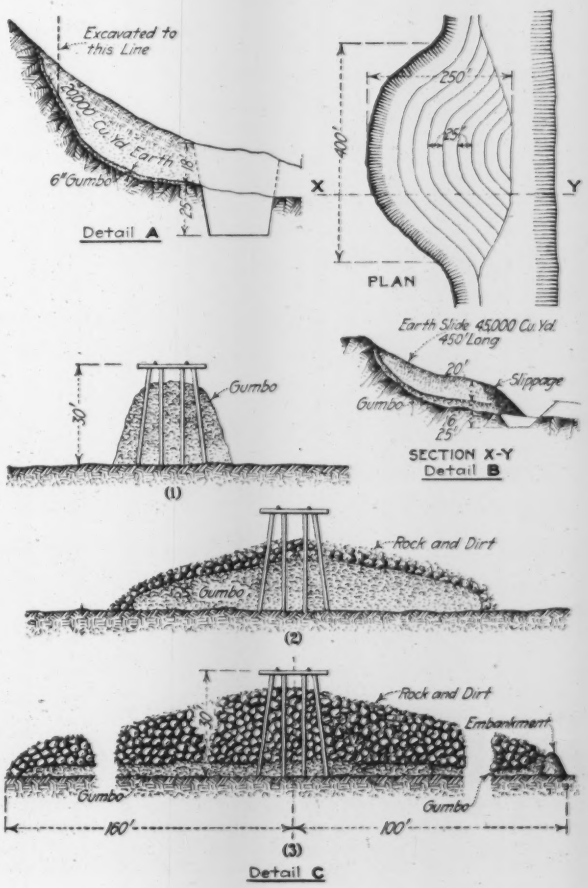


FIG. 3. EXPLANATION OF SLIDES CAUSED BY GUMBO

this work is the same as on the Baltimore & Ohio Southwestern Ry. crossing just described.

The extensive rockwork made blasting in the cuts almost universal on the southern half of the line. Drill holes are put down about 20 ft. and are spaced 10 ft. apart in both directions. Drilling, blasting and excavating proceed simultaneously. The drilling is carried on where the shovel will make its next trip or directly ahead or behind the shovel. The holes are first sprung by shooting 15 to 20 sticks (about 10 lb.) of dynamite. The final or blasting charge averages 1 lb. per cu.yd. of rock. Shooting can continue 100 to 150 ft. ahead of the shovel. Blasting is carried on as closely as possible to the permanent slope line.

The number of shovel trips through typical 60-ft. and 95-ft. rock cuts is shown in Fig. 2. As there was very little water encountered in the rock cuts, the shovel cuts were horizontal instead of being inclined for drainage.

Special Cutting Methods Employed

A "razor edge" hill, which must be cut to a depth of 90 ft., is encountered on Residency 1. The cut was started at grade and will be only 300 ft. long. The broken rock is loaded by hand on mule-drawn cars running on a narrow-gage track. The work is being done by the Perkins Construction Co. and should be finished by the first of the year.

An unusual method is being employed in the case of a cut 1,200 ft. long by 95 ft. deep at maximum. The hill has a hump on its south slope about 400 ft. long. A 6x10-ft. tunnel has been driven through this hump at grade on the slope line (on the left, looking north) and timbered. Holes are put down by churn drills and shot off in 15- to 20-ft. lifts; thus the blasting continues on down as close to the roof of the tunnel as is deemed safe. The aim is to break up the rock all the way down to the roof, but not to break through into the heading. Where the holes do not penetrate deep enough, dobey shots are necessary. In one or two instances the timbering was blown down and the heading had to be cleared of débris. After several shots were fired, cars were run into the headings, the timbering was removed (beginning at the toe of the slope of broken material from shot) and the broken rock above was raked down into the cars.

After all the rock above the heading is cleared away, the cut will be shot to width and a shovel will be put to work. The remainder of the cutting is done as illustrated by Fig. 2. The hump was blasted by a subcontractor for W. H. H. Allen, who in turn is a subcontractor for Rinehart & Dennis. From 6 to 12 men (usually 6) do the work. This method was employed because of the fast progress made by these same men on a similar job elsewhere. It is said that they there removed 6,000 yd. per month. On the present job they are averaging only 2,000 yd., owing to the heavy going.

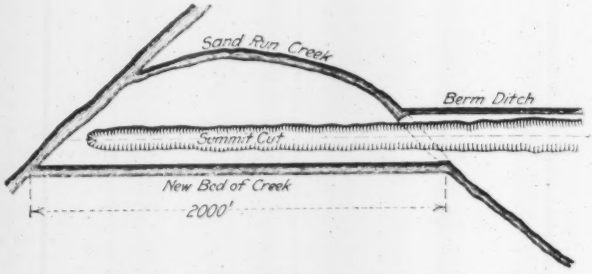


FIG. 4. CREEK DIVERSION TO ALLOW EXCAVATION OF SUMMIT CUT

A revolving steam shovel mounted on caterpillar wheels is a very novel feature, introduced by George Maxfield, a subcontractor on Residency 3. This machine, an 18-B Bucyrus with a 7/8-yd. dipper, is employed on highway-change work. The cuts were many and shallow, and in order to save on moves the shovel was mounted on caterpillars. The scheme has been a success.

A feature of this work is the long hauls in certain instances. There are two 5-mi. hauls and one 5½-mi. Up to 3 mi., 8- to 16-car trains of 4-, 4½-, 5- or 12-yd. dump cars on narrow-gage track, drawn by 20- to 70-ton dinkeys, are employed. On the long hauls stand-

FIG. 5. TRAPPING METHOD OF EXCAVATION—TUNNEL IS DRIVEN; MATERIAL ABOVE IS BLASTED DOWN TO TOP OF TUNNEL, AND TOP TAKEN OFF TO LOAD CARS

ard-gage trains are used. On some of the shorter hauls on Residencies 2 and 3, dump wagons are employed.

An especially difficult movement of equipment was accomplished by the C. W. Marshall Construction Co., of Kansas City, Mo., when it moved its plant from Lucasville on the Norfolk & Western Ry. to the work near Harrisonville, about 9 mi. distant. Three traction engines and a 5-ton White dump-body truck were employed. Almost at the start a 15% grade 700 ft. long was encountered against the movement. The shovel was stripped of its boom, dipper handle and dipper, which were placed on house-moving trucks and drawn up the hill by a traction engine. The shovel proper was placed on heavy rails and sent up under its own steam. The dinkeys were mounted on house-moving trucks and drawn by two traction engines each. The 5½-yd. dump cars were transported by the 5-ton White truck, one at a time. Three trips were made in a day of 10 hr., or a total of 55 mi. per day, 13 days being required for the total movement. The moving gang consisted of 2 foremen, 18 men and 2 teams. The truck is still employed to convey coal for the plant and carries the men and supplies to and from the job.

The Jones-Gray Co., Inc., also performed a difficult feat of plant moving when its outfit was taken from Given, on the Detroit, Toledo & Ironton R.R., to the job, about 7 mi. distant. Temporary track was built. Four locomotives, two steam shovels, one spreader, and three 12-yd. dump cars loaded with blacksmith and machine-shop supplies traveled in one train, all chained together. A shovel was placed on each end of the train facing outward, with the locomotive in the middle. Wire cables around the shovel drums helped the train uphill or eased it down. On heavy grades lines were thrown out by both shovels. Both the shovels and the locomotives traveled under steam on these grades.

The weight of this train made it necessary either to reinforce the bridges along the line or to build temporary crossings of cribwork and stringers. The train took 15 days to reach the job. In addition to this train, there were forty-nine 12-yd. dump cars which were transported in a truck hauled by a 22-hp. Buffalo Pitts traction engine.

The plant of Rinehart & Dennis, Section 2, consists of seven steam shovels, one 80-C Bucyrus, two 70-C Bucyrus, one 65-C Bucyrus, one Marion 61 and two Marions of model 41. Four of these shovels were moved under their own steam from Slocum station on the Baltimore & Ohio Southwestern Ry. from 3 to 6 mi. (with the same system used to move back in cuts), sometimes

FIG. 6. WELL DRILLS WORKING IN STRATIFIED CUT ON RESIDENCY NO. 3

FIG. 7. START OF 45,000-YD. SLIDE IN CUT; SHOVEL CLEARING AWAY

FIG. 8. SHOVEL PARTLY BURIED FOLLOWING BLAST IN SIDEHILL CUT

FIG. 9. TAKING OUT A THROUGH CUT IN ROCK; SHOVEL IN BACKGROUND

coupling two or three of them together. One was moved from Sciotoville along the county road a distance of 6 mi. the same way. The other two were near the work on the Baltimore & Ohio Southwestern and the Norfolk & Western railroads and only required to be moved up on the cuts.

Eight 18- to 60-ton dinkeys and 50 dump cars were moved in over the same route as the shovels. This required moving them up and down grades as high as 20%, which was accomplished with the aid of cables to pull with or hold back. This plant had to be transported across the Little Scioto River. The highway bridge naturally could not be used, and it was necessary to go down the bank some 35 ft., cross in the bed of the stream under the bridge and back up the other side. One of the 60-ton locomotives, a steam shovel and the 18-yd. dump cars now working on the first two cuts north of the Ohio River bridge had to be taken in from the river edge and up a 11% grade on a very sharp curve of probably 50°. This removal was accomplished by first sending the shovel up, pulling itself by cable from drum to hitches on trees. The locomotive was pulled (under steam) by the shovel with a cable from the drum. The dump cars were pulled up by cable.

Four Spreaders Used; Two Creeks Diverted

Four spreaders are used by three different contractors. The first is a western spreader, two are Jordans, and the fourth is a home-made machine built by the Jones-Gray Co., Inc. All four have given excellent service. The two Jordan spreaders are all-steel (except floor) and will handle any rock that the shovel can load on a car. With hand spreading the rock must be broken up into one-man size, which means a lot of additional labor. The spreader works a width of 18 ft. on each side and can work as low as 18 in. below rail. Thus even for a double-track fill the construction track on top does not have to be shifted to one side, as is necessary where shovel gangs are spreading.

Summit Cut is the longest, and the spoil to be removed amounts to 700,000 yd. The deepest point is 55 ft. Along the first 1,500 ft. of the proposed cut, and crossing it, was a small creek, the drainage area of which is about ¾ sq.mi. To cut through the creek it was necessary to divert its course, as shown in Fig. 4. The same problem was met north of Summit Cut, where the drainage area was smaller. In each case the old creek bed served thereafter as a berm ditch, the regular berm ditch connecting with it at each end.

Sliding Ground in Cuts and Fills

A number of slides have been encountered in the various cuts, of which some were of serious proportions. No halfway measures of relief were taken, however, and the slides were overcome by removing all the sliding material in the zone of slippage. Fig. 3 shows the reason for the largest two slides, which occurred on Section 2. In detail A it is seen that a rock pocket was filled with earth about 18 ft. deep. Between the rock and the earth was a 6-in. layer of gumbo, on which the earth slid as soon as the shovel had gone down sufficiently. After the shovel had removed the first lift, a small slide occurred, which increased in proportions as the depth of the cut increased. Finally it became necessary to remove all slipping material, which amounted to about 20,000 yd. This was accomplished in the same general manner as shown in detail C for the second and even larger slide. In the case of the second cut there was also a pocket, but between the earth and the rock was a 16-ft. layer of gumbo, on which the earth slid with great facility. The portion of this sliding stratum that had to be removed by shovel contained about 45,000 yd. of material and was 450 ft. long by 20 ft. in maximum depth.

On the Chesapeake & Ohio Northern work, as elsewhere, gumbo was found to be a poor material for fill. All of it is now wasted. The action of gumbo in a fill is illustrated in detail C, Fig. 3. This fill is on Section 2. As shown, the gumbo held up pretty well almost to the top of the trestle, when it began to subside. Rock and dirt were then dumped upon it, and it continued to

FIG. 10. BUILDING FIRST LIFT OF 90-FT. ROCK FILL IN DAN WHITE HOLLOW

flatten. The gumbo acts as a conveyor and carries the rock down the slope and around the toe. It is hoped that the rocks, which are piled up under the toe, will work in and mix sufficiently with the gumbo to stop the slide eventually. An earth embankment has been built on one side of the toe, as illustrated; against the embankment, rocks and stones have piled up, and the lateral movement seems to have stopped. When the slipping has ceased, a second trestle will be built on top of the fill in order to bring the fill to the full height. The other method of building a high fill is, after the first lift has been made, to jack up the track until full height is attained.

Another poor material that is being wasted resembles quicksand; it contains red, white and blue clay. This material overlies the rock to a depth of about 8 ft. and

carries flowing water. Above this poor material is a 20-ft. stratum of red clay, which is excellent for filling. The wasting of this treacherous material and of the gumbo was not without its difficulties. At one point it was decided to waste into a ravine, moving over the dumping track as the fill widened, but when this was done, the track settled to such an extent that it was unsafe to allow cars upon it. The best scheme for wasting now seems to be dumping on the rock fills. The poor material fills the voids between the rocks and runs down the sides. This scheme provides plenty of space for disposing of the gumbo and from a contracting standpoint has but one disadvantage—it often necessitates long hauls, which of course represent a complete loss to the contractor.

M. J. Caples, vice-president of the Chesapeake & Ohio Northern Ry. and of the Hocking Valley Ry., is in general charge of the work. W. Michel is Chief Engineer. The resident engineers are: H. B. Watters, Sections 1 and 2; W. L. Roller, Section 3; J. W. Holman, Section 4.

Ballasted-Deck Pile Trestles; Kansas City Southern Ry.

Pile trestles built of creosoted timbers are used extensively on the Kansas City Southern Ry. They have ballasted decks to give increased life and reduce the danger of fire. The construction is shown in the accompanying drawing.

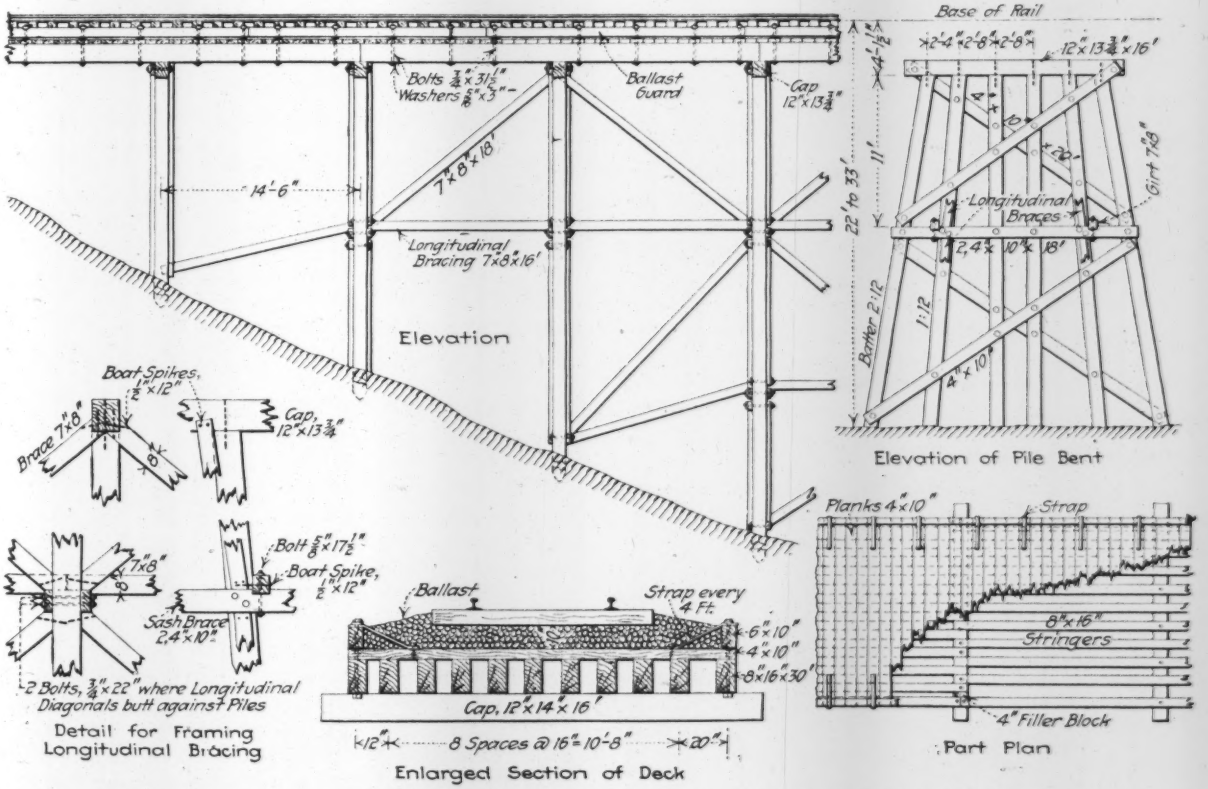
The trestle design is a good example of modern practice in a type of structure that has been a notable feature of American railways since their earliest days. Bents over 22 ft. in height have horizontal sash braces 11 ft. apart with swaybracing between them. Upon the sash braces are bolted girts or horizontal longitudinal timbers. Diagonal longitudinal braces are fitted in each panel, except in those bottom panels where they might form an obstruction to the free passage of drift.

The end bent has five piles, and back of the cap and stringers are three 6x10-in. header planks to hold the end of the roadbed. In trestles having an odd number of panels, one end panel has all its stringers 15 ft. long.

The caps and stringers are sized to 12x13¾ in. and 8x15¾ in. before creosoting. The treated timbers are handled so as to obviate cutting as far as possible, but where they have been cut in framing, the fresh surfaces are given three coats of hot creosote oil. Boltholes bored through are filled with the oil, and the bolts are coated with creosote before being placed. If the hole is not used it is closed with creosoted plugs (after the oil filling). Where spikes are removed, the holes are filled with oil and plugged in the same way.

The ends of the deck stringers are lapped on the caps, except that the outside stringers are fitted together over alternate caps. The amount of gravel required for ballast is about 0.233 cu.yd. per lin.ft. of trestle.

An earlier construction of solid deck (with creosoted timbers) was for fireproofing purposes. This had transverse floor planks laid upon the ordinary packed stringers and covered with 2 in. of chats or washed gravel as a protection against sparks and hot ashes. The rails were spiked to the floor. The new standard trestle design with ballasted floor, as here described, is more satisfactory and gives a better track. It was designed under the direction of C. E. Johnston, Chief Engineer of the Kansas City Southern Ry.



PILE TRESTLE OF CREOSOTED TIMBER, WITH BALLASTED DECK; KANSAS CITY SOUTHERN RY.

Underground-Water Dispute in California Cities

By Kenneth Q. Volk*

A controversy has developed between the cities of Riverside and San Bernardino, Calif., over their respective available water-supplies. San Bernardino is trying to enjoin Riverside from diverting water from the San Bernardino artesian basin. As the amount of available water is the measure and the limit of the development of these semiarid regions, this supply is of vital interest to these communities.

The runoff from 425 sq.mi. of high mountain drainage area discharges onto absorbent gravels in the eastern end of the San Bernardino valley, and these waters percolating seaward come in contact with what is known as the Bunker Hill dike, near Colton, where artesian conditions are produced. The artesian cap in this instance is a clay of more or less continuous nature.

The people of San Bernardino County obtain their local water-supply both for the town and for agricultural purposes by drilling wells through this clay cap and directly applying water to their lands and pumping it to their houses. In years gone by these artesian pressures at San Bernardino were sufficient to furnish domestic pressure throughout the house, but as the agricultural development has progressed these pressures have decreased. There are now 50,000 people and nearly 80,000 acres of land served from this artesian basin and the surrounding streams which feed it.

The laws with reference to the ownership of underground water have not been completely developed and fixed in California. The old rulings have been to the effect that waters percolating underground belonged to the land, and the owner of the land could extract and dispose of them at will. The later rulings of the California courts, however, have been to the effect that this can only be done as long as it is not a trespass on the owner's grounds; in other words, that a man could not buy a 100-ft. lot, start a pumping plant and not only extract water from the gravels under his own land, but also in an invidious way withdraw water from his neighbor's land. These conditions are modified and amended by what are termed the prescriptive rights. Where water has been developed for a period of five years without protest, the right becomes fixed and one of the most perfect that can be obtained to water in California.

This particular controversy is to determine permanently the respective rights of these two counties to the underground waters of this San Bernardino valley. It has involved enormous engineering and geological studies, and the plaintiff, which was the City of San Bernardino, took six weeks' time in presenting its case, from the middle of June to the first of August. The case then was postponed with the hope that a compromise agreement could be arrived at, and efforts have been made by representatives of the different parties at interest to accomplish this, but without avail. It now appears that the case will again go to trial in the early part of April.

Extensive studies have been made of the character and extent of the artesian gravels, the rise and fall of the water plane, the limitations of the artesian areas and available water-supply, as well as the legal rights.

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Reducing Hillside Street Grade from 29 to 16 Per Cent.

On account of the fine views obtainable from their slopes some of San Francisco's hills are popular residence sites. Unfortunately, however, little thought was given to topography by the surveyor who in 1849 first laid out the rectangular street system, which results in grades of 55% on some of these side-hill streets. To eliminate at a minimum cost as many as possible of these excessive grades has been the recent policy of the city. No standard plan has been adopted, but a separate study is made of each case.

An interesting regrade problem recently undertaken at the request of the property owners affected is the redesign of Larkin St. between Chestnut and Francisco St., which reduces the grade from 29% to a maximum of 16%. The cost of the proposed improvement will be paid for by the owners having frontage on the streets to be regraded. On Larkin St. $16.31 per front foot will be assessed against the property on each side of the street, and on Francisco St. the rate will be $20.29 per front foot. Present property values on both these streets are now in the neighborhood of $115 per front foot. The regrade will probably cause an advance in value to $200.

Regrades on Larkin Street

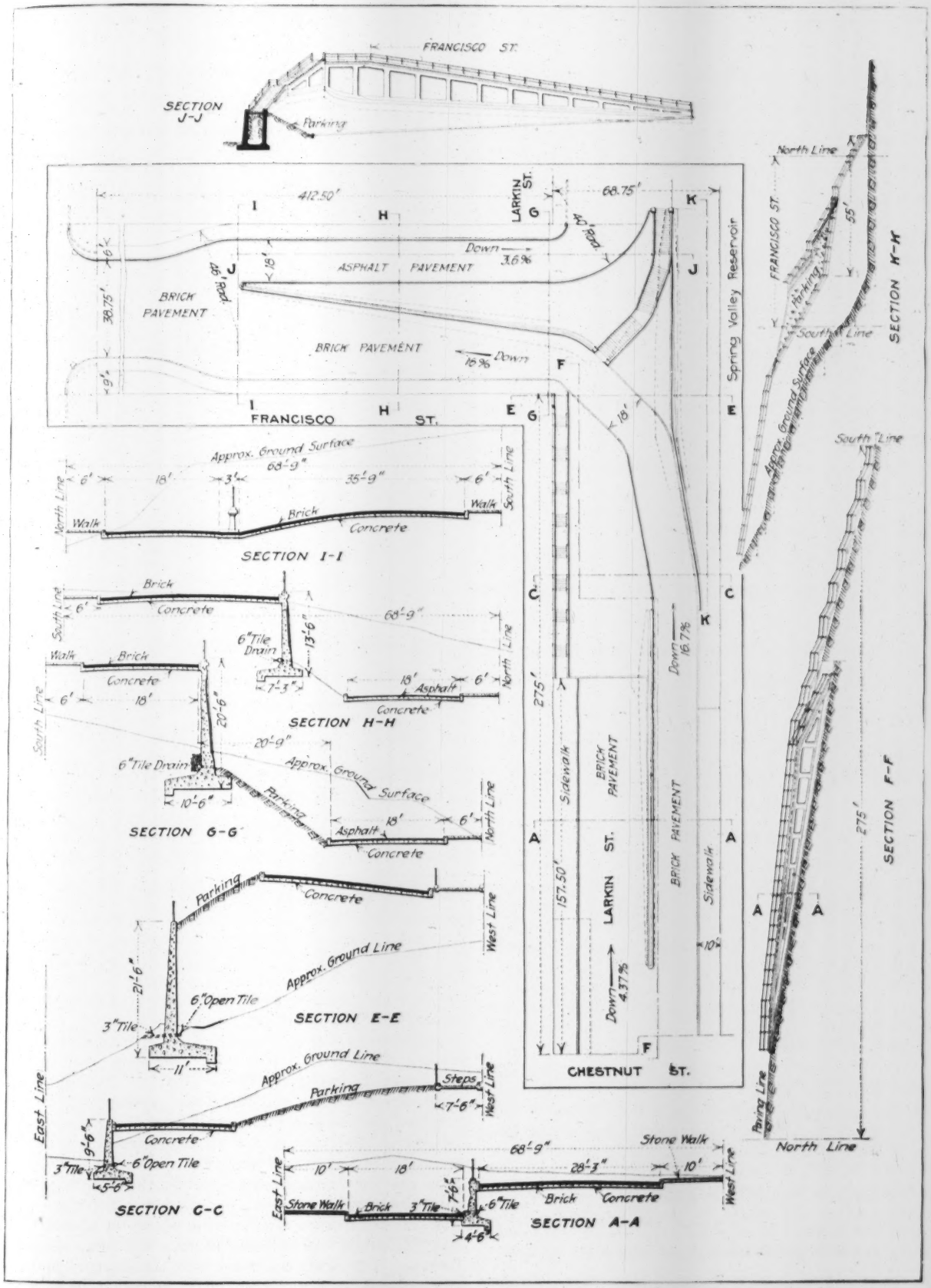
For a width of 28 ft. the east side of Larkin St. (see accompanying plan) between Chestnut and Francisco St. will be cut down to descend on a uniform grade of 16.07%. On this incline will be an 18-ft. vitrified-brick roadway and a 10-ft concrete sidewalk. This strip will be separated from the west side of the street by a reinforced-concrete retaining wall, and near the bottom of the hill will turn into the south side of Francisco St.

The west side of Larkin St., which will have a higher elevation than the east side, will descend on grades varying from 4.37 to 10.92% for a distance of 157.5 ft., where a vitrified-brick pavement 28.25 ft. wide will terminate in a parked slope. At its termination the west roadway is 12 ft. higher than the east roadway directly opposite. The west sidewalk will continue down the west side in a series of steps to the level of the east roadway at Francisco St.

Turning westerly the descending east roadway will continue on a 16% grade along the south side of Francisco St. for a distance of 127.5 ft., separated from the north side of the street by a reinforced-concrete retaining wall. Around the westerly end of this wall the roadway turns through 180° and descends easterly along the north side of Francisco St. on a 3.6% grade back to Larkin St., into which it turns northerly.

The sewerage arrangement consists of two 8-in. pipe sewers, dead-ended at Chestnut St., descending under each of the pavements on Larkin St. to a point opposite the beginning of the stairway. Here the westerly 8-in. sewer is connected to the easterly 8-in. sewer by a 12-in. pipe. The combined sewers are a 12-in. pipe turning to the west on Francisco St. The underdrains of each retaining wall are connected to these sewers through catchbasins. In profile the sewers are parallel to the new street surfaces.

The contract for this construction work was awarded to F. Rolandi at an estimated cost of approximately $30,000. The construction will be done under the direction of M. M. O'Shaughnessy, City Engineer.



PLAN, ELEVATIONS AND CROSS-SECTIONS OF THE REDESIGN OF PART OF LARKIN ST., SAN FRANCISCO

Eight Years of Imhoff Tank Design and Operation--I*

By Karl Imhoff†

SYNOPSIS—The engineer who developed the form of two-story sewage-settling tank or modified septic tank bearing his name reviews eight years' German and American experience in design and operation of Imhoff tanks. The translator, who worked with Dr. Imhoff in Germany and was subsequently engaged on the design of Imhoff tanks for Baltimore, Md., has added a number of supplementary notes.

Form and Dimensions of Tank

The form of sewage-settling and sludge-digesting device known in Germany as Emscherbrunnen and in America as Imhoff tanks was first introduced eight years ago. Today there are many municipalities making use of it. In Germany there are 170 plants designed to treat the sewage of 3,000,000 people. Of these plants 130 are in operation and 40 are under construction. In the United States 70 plants, to care for 600,000 people, have been designed, and over half of these are in operation.

The following four questions are pertinent: (1) What form of tank has given the best results? (2) What should be the dimensions of the plant? (3) How should the plant be operated? (4) What performance may be expected?

What Form of Tank Has Given the Best Results?

In order to discuss the best form of tank it will be necessary to describe the most important forms that have been used.

Fig. 1 shows the original design of Imhoff tank: A horizontal-flow sedimentation chamber situated above one or more sludge-digestion chambers. The invert of the sedimentation chamber consists of thin partition walls which serve to direct the sewage flow but which are not required to withstand water pressure. These partition walls are inclined at such an angle that the sludge which settles out from the sewage may not form harmful deposits upon the walls but must glide off and pass into the sludge chamber through slot *a*, situated at the lowest point of the sedimentation section. This slot is overlapped so that no gas bubbles or sludge particles rising in a vertical direction in the sludge chamber may escape into the settling chamber.

The inflow of sewage into the settling chamber takes place through a gate. The distribution of the sewage is effected by a shallow baffle having a penetration of 12 in. A like baffle is situated at the effluent end of the tank. In order to maintain an approximately constant water level in the tank the outflow of sewage should take place over as long a weir as possible, which should therefore extend across the whole width of the sedimentation chamber. If there is more than one sludge chamber beneath the settling chamber the direction of the sewage flow through the tank must be reversed about once every three weeks. The bypass channel *b* is provided for this purpose.

The sludge in the sludge chamber decomposes biologically, so that after a time it loses its objectionable character. It can then be discharged through the sludge pipe *c* and conveyed either by gravity or by means of pumps to the sludge-drying bed *d*. The liquid sludge is black and has an odor somewhat resembling that of tar or rubber. A layer of sludge 12 in. deep will reduce in drying to 8 in. in about one week and has then many of the characteristics of garden earth or humus.

What is most important is that the processes of decomposition in the tank be limited to the sludge. Only then will these processes take place without obnoxious odors. It is necessary therefore to make sure that no flow of sewage takes place through the sludge-digestion chamber. As soon as appreciable amounts of sewage are permitted to pass through it such sewage decomposes and develops putrid gases similar to those developed in the well-known septic tank.

The removal of settleable solids takes place in the settling chamber entirely independently of the processes of decomposition taking place in the sludge chamber. The extent to which settleable solids are removed depends upon the time which the sewage consumes in flowing through the tank. If this time, known as the detention period, be not made too long the sewage will be practically as fresh on leaving the tank as it was when it entered.

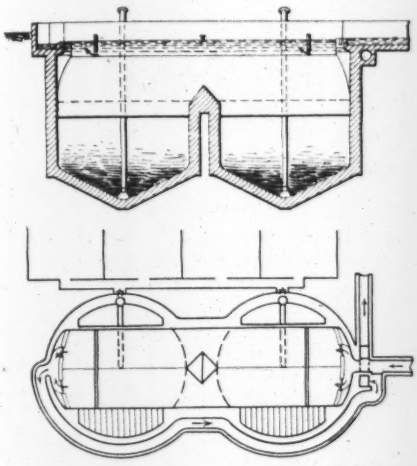
Adding an Element of Economy

This form of Imhoff tank is the simplest as well as the oldest, and after the lapse of eight years it must still be designated as the best. Only in one particular is it possible to add an element of economy. Operation experience has shown that the surface of the sludge should not be allowed to rise above the lowest point *a*, Fig. 2, of the inclined diaphragm. This fact limits the capacity of the sludge chamber. It is possible to increase this capacity considerably if instead of designing the tank in accordance with form *B* it be designed in accordance with form *A*. It is necessary in any case to support the inclined walls by means of a longitudinal beam; this beam may just as easily be given the section shown in design *A* and be thereby made to act as a protecting gas deflector for the two slots which have now been formed from the original single slot. This form of construction was first used in 1907.

Many variations of the original form of tank have been attempted, but they cannot all be described here. For example, in the case of very wide tanks the sedimentation chamber has been designed with three slots (Goerlitz) in order to prevent too deep a penetration into the sludge chamber. In America, where the sewage contains relatively less settleable solids, the settling chamber becomes large when compared with the sludge chamber; the natural tendency therefore is to design it as wide as possible and to provide it with more slots (Fig. 3).

*Translated by Leslie C. Frank, Sanitary Engineer, United States Public Health Service, Washington, D. C.

†Division Engineer, Emscher Federation, Essen, Germany.



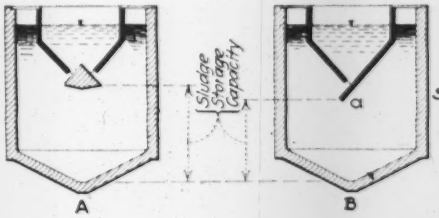


FIG. 2. EFFECT OF DESIGN ON SLUDGE-STORAGE CAPACITY (A & B)

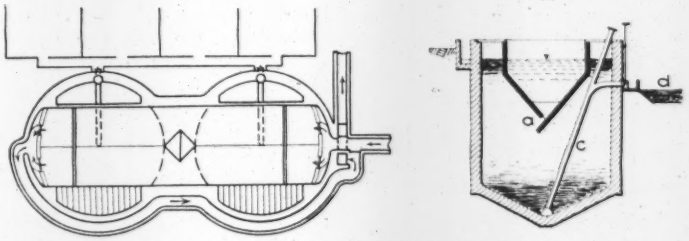
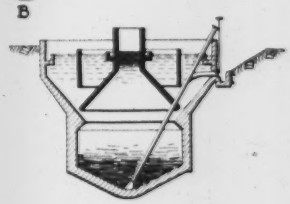
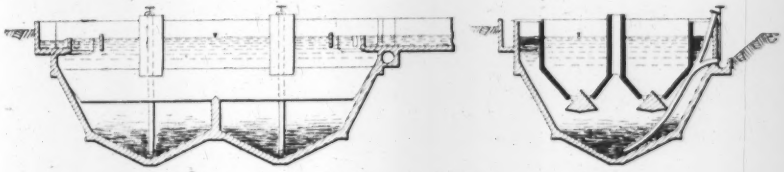


FIG. 1. ORIGINAL DESIGN OF IMHOFF TANK





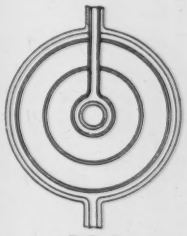


FIG. 5. DOWNWARD AND UPWARD FLOW TANK

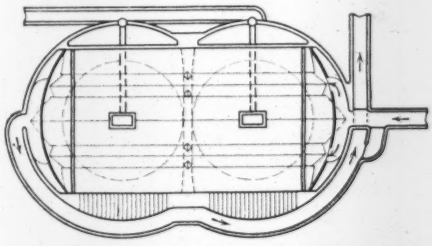


FIG. 3. FOR WEAK SEWAGE: LARGE SETTLING CHAMBER AND SMALL SLUDGE CHAMBER

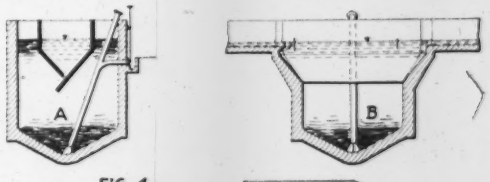


FIG. 4. HORIZONTAL EXTENSION OF SETTLING CHAMBER (A, B & C)

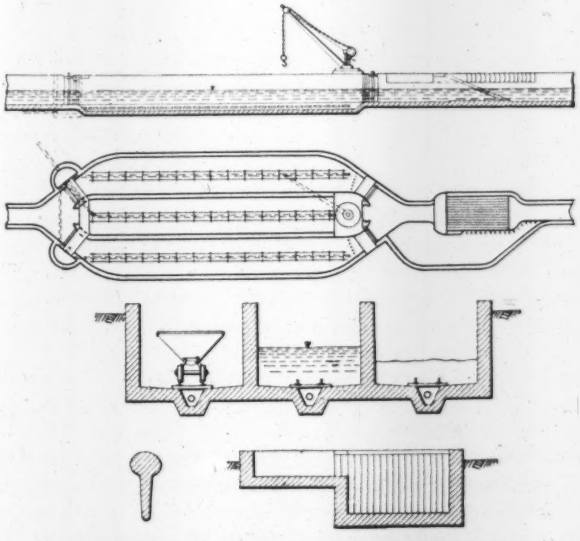


FIG. 6. COARSE SCREEN AND GRIT CHAMBER

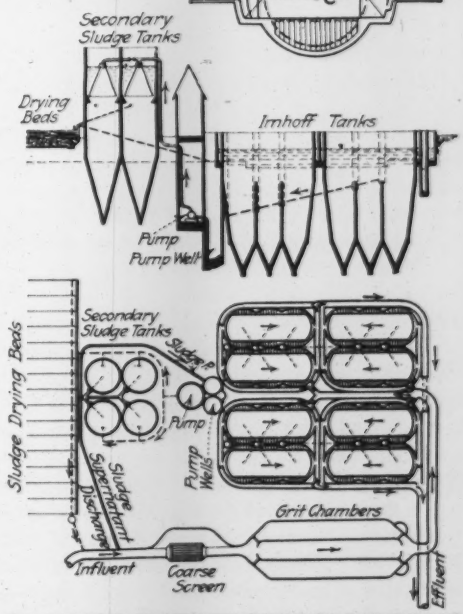


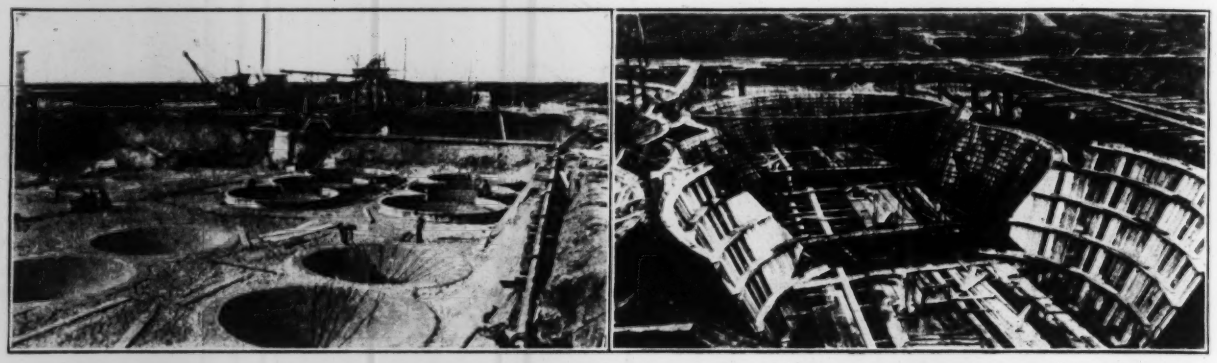
FIG. 7. IMHOFF PLANT WITH SECONDARY SLUDGE TANKS

FIGS. 1 TO 7. DEVELOPMENT OF DESIGN OF IMHOFF TANK FOR SEWAGE CLARIFICATION AND SLUDGE DIGESTION

For the same reason it becomes desirable in many cases to extend the sedimentation chamber horizontally beyond the sludge chamber (Fig. 4). Not more than two or three sludge chambers should be connected with one settling chamber, but here hard and fast rules cannot be given.

Round Chambers Generally Cheaper

In all of the accompanying illustrations only round sludge chambers are shown. It is not intended thereby to designate round sludge chambers as the best for all cases. In America many rectangular plants have been built. The American engineers justify this by the statement that contractors find difficulty in constructing round forms for reinforced concrete and therefore exact an accordingly higher price. I believe, nevertheless, that round tanks are cheaper, because the walls may be designed upon the basis of hoop tension only, since they are not subjected to bending moment. Consequently the wall sections are relatively much thinner and the amount of steel required relatively much less. I have never built other than round sludge chambers, since a cylinder would seem to be the natural shape for any water container. The round shape also makes it possible to construct the tanks in either of two ways: To sink them as caissons, or to use open-pit construction. In the first plants constructed in Germany the tanks were sunk as caissons, and this method of construction is to be recommended wherever the ground is soft. The open-pit form of construction is often possible, of course. (See the views, Figs. 8 to 13, for examples of the two forms of construction.)



FIGS. 8 AND 9. OPEN-PIT CONSTRUCTION AND FORMING AT MEIDERICH-HAMBORN, GERMANY
Left—Sludge sumps. Right—Flare forms

Regardless of how the tanks are designed or constructed they must at least be sufficiently strong to withstand the inner water pressure without reliance on any exterior support. It is very seldom indeed that it is possible to depend for support upon the ground material adjacent to the tank walls.

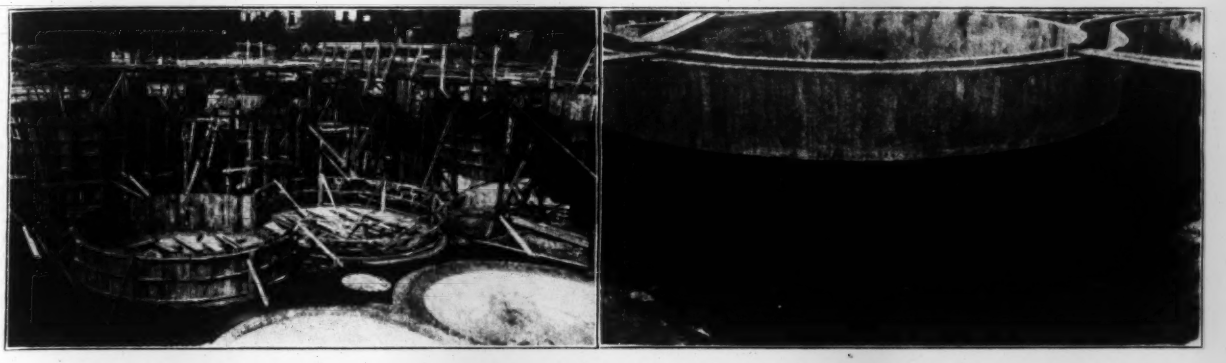
All examples of Imhoff tanks thus far shown have the sewage flow taking place through them in a horizontal direction Experience has shown that this is the best method. Plants in which the direction of sewage flow is downward and upward have not given such good results. Their description in this paper will therefore be made brief. Such tanks as have been built for downward and upward sewage flow have consisted of individual round units. In them the sewage is led to the center, flows out radially and downward under a deeply penetrating baffle and then upward to the circumferential effluent channel (Fig. 5).

The advantage of this design is that it adapts itself admirably to the circular form of construction. Experience in many plants of the Emscher district has shown, however, that the removal of settleable solids is less efficient in this form of tank than in the horizontal-flow design. The suspended solids have little opportunity of being removed from the sewage during its downward course, and the upward direction assumed by the sewage, once the baffle has been passed, tends to carry with it some of the lighter settleable solids and to permit them to pass off with the effluent. The downward-and-upward design of plant is totally unadapted for sewage coming from "combined systems" if the velocity of flow through the plant during times of storm is expected to be increased. The disadvantages of the downward-and-upward system in the case of the separate system of sewerage are relatively less. I no longer recommend this type of tank under any circumstances. If it is desired to convert such tanks for horizontal flow, it is merely necessary to remove the deep baffle and to lead the sewage through the tanks in a radial-horizontal direction, substituting for the deep baffle a shallow scum ring near the outer circumference. Tanks of this latter shape have been designed and constructed for the city of Baltimore. It is possible to modify these tanks still further by leading the sewage through them from one side to the other rather than radially. Plants of this kind have been constructed in Germany.

Deep tanks are to be preferred to shallow tanks because in them the decomposition of the sludge is improved. This is so because in these deeper tanks the temperature is maintained more uniformly and because the stirring action of the rising gas bubbles is more intense. Because of the relatively better sludge decomposition the sludge chambers of the deeper tanks need not be designed so large. In addition the sludge-drying beds may be designed smaller because the sludge from the deeper tanks contains more gas and dries more easily.

[To the advantages of depth of tank may be added the possible advantages of depth of sludge layer. Sludge flakes are probably compressible and may be likened to very small sponges. If a piece of wire screen were made to cover a mass of water-soaked sponges in a container and the screen then loaded with weights the

sponges would compress and a water layer appear above the screen. The moisture content of the sponges would naturally thereby be reduced. In like manner any horizontal plane in a mass of sludge may be considered as being occupied by a screen and the mass of heavier-than-water sludge above this plane as equivalent to the weights. It seems probable then that if sludge particles are both compressible and heavier than water the depth of sludge layer will have an important influence upon the average moisture content of the sludge, or the deeper the layer the lower the average moisture content. This may afford another reason why with deep tanks the sludge chamber need not be designed so large as with shallow tanks. The same factor will also influence the required area of sludge-drying beds.—Translator.]



FIGS. 10 AND 11. IMHOFF TANKS UNDER CONSTRUCTION BY OPEN-PIT METHOD AT STEELE, GERMANY
Left—Sludge-chamber sump completed in front; form work for sludge chamber in middle and in rear. Right—Forms removed from top of tank

In discussing Imhoff sewage-treatment plants it will be pertinent to consider those parts auxiliary to the tanks themselves. The simplest designs have here also proven to be the best.

Coarse Screens and Grit Chambers

Coarse screens are absolutely necessary. The function of these screens should not be confused with that of so-called fine screens. The function of the coarse screens is not to remove as much of the suspended matter as possible, which is true of fine screens, but instead to remove only such coarse matters as would tend to clog the Imhoff tank sludge pipes. Above all it is highly desirable that fecal matter be not retained on these screens. Coarse screens therefore should have a clearance of about 2 in. They should be placed in an inclined position and the sewer cross-section at this point should be increased somewhat in width, but not much. The coarse screens should be cleaned by hand from time to time. Only in the case of very small plants which are close to the source of the sewage and at which the fecal matters arrive in an undivided state will it occasionally prove advisable to install a screen with a smaller clearance than above, on which these undivided fecal matters may be retained and broken up so that they will not float for too long a time upon the surface of the tank.

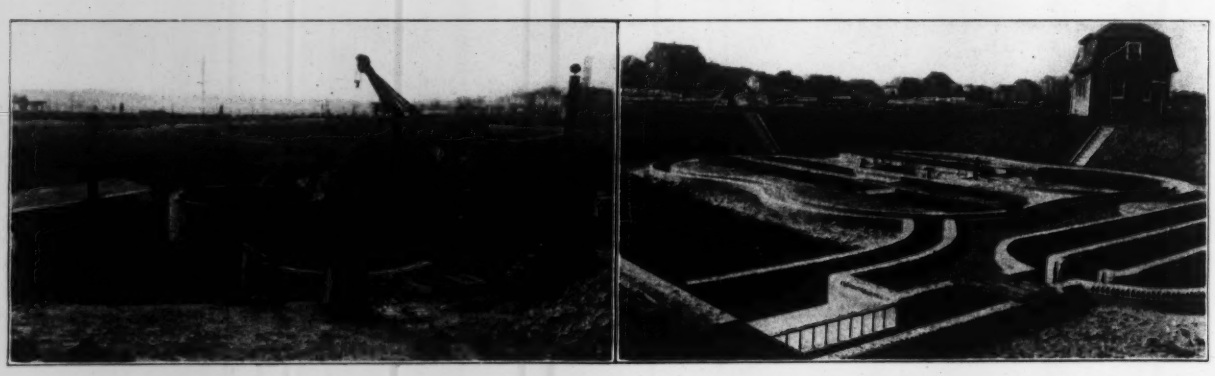
Grit chambers will be required in large plants that receive sewage from a combined system. In the case of grit chambers, as in the case of coarse screens, it is undesirable to retain appreciable amounts of organic matter. Its presence would induce difficulties in the subsequent handling of the grit. It should in fact be the aim to retain as nearly as possible only pure sand or grit. To this end a velocity of about 8 to 12 in. per sec. should be maintained in all parts of the chamber. The grit chamber ought not to be deep. It forms, so to speak, simply a modified part of the sewer, in which the velocity of flow has been reduced to below the grit-transporting point. The length of the grit chamber depends on the amount of grit which it is expected will be received during the interval between cleanings, say three days. The grit chamber may most advantageously be constructed in three units. To clean one of these units it is necessary to take it out of operation and drain the retained grit by means of the underdrainage system provided in the design (Fig. 6). The grit may then be removed by hand or with the aid of a crane.

The system of sludge pipes provided for the sludge chambers must include a pipe and a gate for each chamber, so that it will be possible to discharge from any chamber independently. The pipe should preferably discharge directly into an open channel, so that the attendant may be able to observe the sludge as it comes from the tank. The hydraulic gradient which should be provided for closed sludge pipes is about 12% and for open channels 2½%. It is advisable to include in the bottom of the sludge chamber a water-agitation system. This system should not be connected permanently with the water mains as it will be used but seldom if at all. The system is provided merely so that it may be possible, if necessary, to agitate the sludge in order to promote its decomposition. If the operation of the tank shows that such agitation is necessary the water-agitation system may be connected with the main by means of a hose.

The sludge beds should consist of a comparatively large number of independent units, so that in drying the sludge some of these units may be receiving fresh sludge while others are drying and while still others are having the dried sludge removed. The sludge-drying beds are provided with an underdrainage system that is covered with a porous material such as slag or gravel. On this gravel must be placed 1½ or 2 in. of sand or some other equally fine material. This layer of fine material must be renewed from time to time because each removal of dried sludge includes a certain amount of the sand layer. Such renewal of fine material is provided for in the plants of the Emschergenossenschaft by making use of the grit recovered from the grit chamber. Each drying bed is provided with a narrow-gage track

placed on a level with the upper surface of the bed. These tracks permit the use of sludge cars and thereby greatly facilitate the removal of the sludge.

In the first Imhoff sewage plants the pumping of sludge was avoided as much as possible because the drying of the sludge is promoted by its gas content, and pumping tends to remove this gas content. This difficulty has since been obviated. In the new plants of the Emscher district the pumps are augmented by a secondary sludge tank. This tank is of such size as to provide for the sludge a detention period of about one week—which will permit the necessary gases to form. From the secondary tank the sludge flows by gravity to the sludge bed in the same condition, so far as its drying characteristics are concerned, as if it had never been pumped (Fig. 7). In many of the more recent plants



FIGS. 12 AND 13. PROGRESS VIEW AND COMPLETED IMHOFF PLANT AT WANNE-NORD, GERMANY
Left—Caisson type of construction. Right—Completed plant, showing screen chamber and grit chamber in front and tanks at rear

the secondary sludge tank has been made larger than above indicated so as to permit the sludge chambers under the settling chambers to be made smaller. This is simply equivalent to providing part of the required sludge-digestion capacity under the settling chambers and another part above the sludge-drying beds.

What Should Be the Dimensions of the Plant?

The settling chamber of an Imhoff tank should be designed like any plain sedimentation chamber. It is necessary to determine first of all what should be the detention period. This depends upon the character of the sewage and upon the degree of sedimentation required. Within limits velocity of flow is not significant. In the settling chamber, if it be properly designed, the process takes place in precisely the same manner as in any quiescent container. It is necessary only to imagine the quiescent container filled with sewage and passing through the chamber at the same velocity as the other sewage in the chamber. It is evident that the velocity of the sewage body as a whole in a horizontal direction, so long as it is not a swirling velocity, has no influence upon the process of sedimentation.

For normal municipal sewage the practice in Germany is to allow a sedimentation period of 1½ to 2 hr. If the sewage contain trade wastes of a nature to promote the process of settling, a period of 45 to 60 min. will suffice. To increase the sedimentation period beyond 2 to 3 hr. I believe to be uneconomical.

[There is very little practical difference between a nominal detention period of 1.5 hr. as used in Germany and a nominal detention period of 2 hr. as used in the United States. The German design assumes that the daily flow takes place in 14 hr., and the American design that it takes place in 18 hr. On this basis the German settling chamber would be equal in size to $1.5 \div 14$, and the American settling chamber to $2 \div 18$ times the daily sewage flow. Per volume of flow the American settling chamber is therefore only 3.5% larger than the German settling chamber. Inasmuch as the 14- and 18-hr. assumptions in Germany and the United States are both arbitrary it would seem simpler to use mean detention period instead of nominal detention period and to say that the mean detention period based on the average daily sewage flow is in most cases in both the United States and Germany about 2.6 hr.—Translator.]

If it be assumed now that the detention period has been determined, the settling chamber is then made of such a size that this detention period will be realized during times of normal dry-weather flow. By normal dry-weather flow is not meant here the maximum flow of the day, but something less than the peak load. In general it is good practice to regard as a normal hourly flow for American conditions one-eighteenth of the total daily flow. In Germany the corresponding figure is one-fourteenth. If, for example, the population of a given town is 10,000 and the sewage flow 50 gal. per head per day, the total daily flow is 500,000 gal. On the above assumption the normal hourly flow is accordingly one-eighteenth of 500,000, or 27,700 gal. If it is desired to give a settling period of 2 hr. the sedimentation chamber must then have a capacity of $2 \times 27{,}700 = 55{,}400$ gal. During times of storm flow the detention period is of course much less. By detention period here is always meant the computed detention period. The actual detention for part of the sewage is naturally less because of short-circuiting; it can be determined by means of dyes placed in the influent.

Dimensions or Capacity of Sludge Chambers

The dimensioning of the sludge chamber is less simple. For this purpose it is theoretically necessary to know how much sludge is to be expected, how far this sludge will be reduced in volume by decomposition, how long it should remain in the tank and how much space should be provided as reserve space for the accumulation of sludge during such time as it is impossible to discharge any. Practically, however, it avails little to attempt to estimate each of these individual factors in order to obtain the final result. The final result might as well

be estimated at once. Sufficient practical experience is now available to estimate the number of cubic feet of sludge capacity that will be necessary per person connected with the plant. The values given in Table 1 are to be recommended.

TABLE 1. RECOMMENDED SLUDGE CAPACITY FOR GERMANY AND THE UNITED STATES
(Cu.Ft. per Head of Population)

	Germany		America—Northern States with Long Winters	
	Separate System	Combined System*	Separate System	Combined System*
Small plants, less than 5,000 population	1.6	2.4	2.4	3.6
Normal city plants whose sewage contains some trade wastes	0.8	1.2	1.2	1.8
City plants whose sewage contains an abnormal amount of sludge-forming trade wastes	1.2	1.8	1.8	2.7

*Presupposes the use of a grit chamber.

With this table it will be possible to estimate the correct value for any particular case. Naturally a considerable latitude is assumed. Deep sludge tanks may require less than the above figures, and shallow sludge tanks more. Plants in northern climates where long winters obtain must be designed larger, and those in southern climates smaller. There are plants whose sludge chambers are designed only half as large as the figures given in the table, and which, with intelligent operation, nevertheless yield very good results. In general it is better to be generous rather than sparing in designing sludge chambers. This should particularly be observed for American conditions because there the greater portion of the cost lies in the sedimentation chamber, which is relatively much larger than the sludge chamber because of the much larger volume of sewage flow per capita.

The volume of wet decomposed sludge that is to be expected for the separate system of sewerage is 0.1 liter per head per day, and for the combined system 0.15 liter per head per day. An additional 0.15 liter or more per day will have to be allowed for cities whose sewage contains much sludge-forming trade wastes. With the assumption that the sludge-drying bed is filled each time to a depth of 12 in. with wet sludge, and that the number of times any one sludge bed is filled during the year is 15, the values shown in Table 2 will hold in determining the size of required sludge-bed area.

TABLE 2. SLUDGE VOLUMES AND SLUDGE-DRYING BED AREAS

	Ordinary City Sewage Containing Some Trade Wastes		City Sewage Containing an Abnormally Large Quantity of Sludge-Forming Trade Wastes	
	Separate System	Combined System	Separate System	Combined System
Volume of wet sludge in liters per head per day	0.1	0.15	0.25	0.3
Area of sludge-drying bed in sq.ft. per inhabitant	0.2	0.3	0.5	0.6

As an average value for the sludge-bed area ⅓ sq.ft. per inhabitant may be kept in mind. Naturally the actual area of sludge-drying beds required will depend largely upon the manner of operation and upon the individual conditions obtaining. Small plants in which it is not possible to discharge sludge so often will require a larger sludge-drying area. Large plants where the operation can be relied upon as being extremely careful and intelligent may require less than the above figures. The sludge beds in the Emscher district of Germany are smaller than the values given.

Model City-Planning Board

A city-planning board to pass upon all proposed actions affecting the plan of the city is a feature of the model commission-manager city charter now being given its finishing touches by the committee on municipal program of the National Municipal League. The city-planning article of the charter, as adopted by the committee on Dec. 27, 1915, is in part as follows:

There shall be a city-planning board of five members consisting of the director of public works and utilities and two citizen members chosen because of their knowledge of city planning. (In larger cities having a director of utilities a board of five members consisting of the directors of public works and utilities and two citizen members is recommended.) It shall be the duty of the board to keep itself informed of the progress of city planning in this and other countries, to make studies and recommendations for the improvement of the plan of the city with a view to the present and future movement of traffic, the convenience, amenity, health, recreation, general welfare and other needs of the city dependent on the city plan; to consider and report upon the design and their relations to the city plan of all new public ways, lands, buildings and bridges and all other public places or structures, as well as all additions to or alterations in those already existing and the layout or plotting of new subdivisions of the city or of territory adjacent to or near the city.

All acts of the council or of any other branch of the city government affecting the city plan shall be submitted by the council to the board for recommendations.

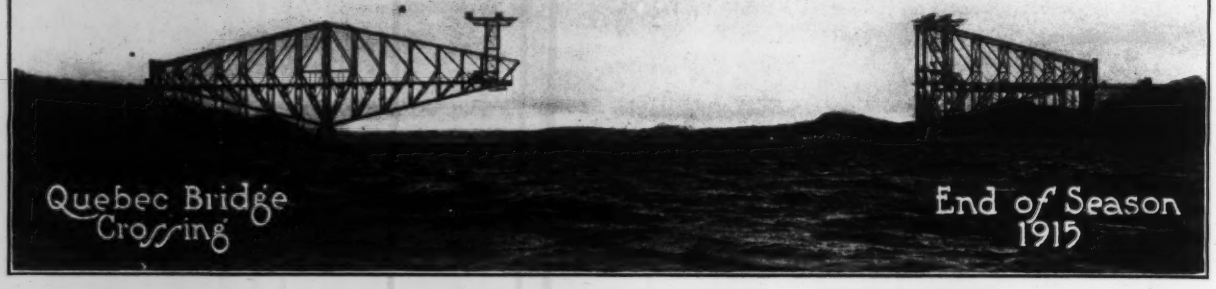
Any matter referred by the council to the board shall be acted upon by the board within 30 days of the date of reference, unless a longer or shorter period is specified. No action by the council involving any points hereinbefore set forth shall be legal and binding until it has been referred to the board and until the recommendations of the board thereon have been accepted or rejected by the council.

The board shall submit to the council an annual report summarizing the activities of the board for the fiscal year, the recommendations made by it to the council during the year and the action of the council during the year on any and all recommendations made by the board in that or former years. The annual report of the board shall also contain a program for improvements to the city plan year by year during at least the coming three years, with estimates of the cost thereof and recommendations as to how the cost shall be met.

The board shall appoint as secretary a person of skill and experience in city planning and may employ consulting city-planning experts as need may arise. The city engineer shall serve as chief engineer of the board, and it shall be his particular duty to make recommendations designed to bring all the engineering works of the city into harmony as parts of one comprehensive plan. The executive health officer of the city shall advise the board from time to time of any municipal improvements within the scope of the board which in his opinion would improve the healthfulness of the city. The board shall have power to call upon any branch of the city government at any time for information and advice which in the opinion of the board will insure the efficiency of its work.

Quebec Bridge Erection: Progress In 1915

By H. P. Borden*



Very satisfactory progress was made during the past season on the erection of the Quebec Bridge. When the work of erection ceased, about the middle of November, the north anchor and cantilever arms had been completely erected, and the south anchor arm out to and including the main post, with the exception of a certain amount of riveting in the latter, which will be finished next season.

Work on the North Shore

Work on the north side of the river proceeded during the summer very closely in accordance with the program mapped out at the beginning of the season. A start on the erection of the steel was made about the middle of April, and by the first of June the anchor arm had been completed and the main post erected and almost entirely riveted. From this point on, the erection of the cantilever arm proceeded without any hitch until completion.

Table 1 shows the progress of the work by dates and the approximate amount of steel erected. The maximum material erected by one traveler in one day was 670 tons. There were several days in which the tonnage exceeded 600. In the erection of the main post an average of over 350 tons a day was maintained. July was the biggest month, 4,200 tons being erected. This refers to one traveler and in each case on the south shore.

The ease with which the enormous members were put in place and fitted was almost phenomenal, practically no delay whatever being occasioned at any of the joints. Owing to the facts that very great care was taken in the shop during the course of manufacture and that all field splices were completely assembled and reamed out in place, no shop errors had to be corrected in the field, with the result that no time was lost in this manner.

TABLE 1. PROGRESS OF ERECTION OF CANTILEVER ARM, QUEBEC BRIDGE

Traveler at Panel Point	Date	Days Required, Each Move	Days Required, Full Panel	Approximate Tons of Steel Erected
Main post 14–16	June 5	.. ..	..	....
15	June 17	12 Subpanel	..	950
14	July 13	26	38	1,993
13	July 19	6 Subpanel	..	828
12	Aug. 11	23	29	1,732
11	Aug. 16	5 Subpanel	..	452
10	Sept. 2	17	22	1,450
9	Sept. 8	6 Subpanel	..	447
8	Sept. 18	10	16	968
7	Sept. 20	2 Subpanel	..	493
6	Sept. 30	10	12	747
5	Oct. 4	4 Subpanel	..	288
4	Oct. 9	5	9	538
Last move 2	Oct. 25	16	11	610
0	.....	.. ..	..	636

Accurate measurements were taken as each panel was erected, to determine whether the cantilever arm was in its theoretical position. Center marks were placed on each floorbeam before assembling, and it has been found that these marks were in perfect alignment from the end of the cantilever arm to the anchor pier.

Lifting of the North Anchor Arm

The total dead weight of the cantilever and anchor arms is 24,000,000 lb. and 26,000,000 lb. respectively, exclusive in both cases of the main post over the pier. The erection traveler under working conditions weighs approximately 1,000 tons. The first effect of the cantilevering out was noticed from a lifting movement in the anchor-arm truss when the traveler was at panel point 11, just after the bottom-chord section CL10-11 had been put in place. Subsequent movements of this truss are shown in Table 2, which gives the vertical movement of the bottom chords which were supported on steel falsework. These chords did not actually lift off the falsework until a vertical movement in the center of over 1 in. had taken place.

TABLE 2. VERTICAL MOVEMENTS, IN INCHES, OF NORTH ANCHOR ARM, QUEBEC BRIDGE

	Panel Points 12	10	8	6	4	2
Aug. 19	0	1/16	3/16	3/16	0	0
Aug. 24	1/16	1/16	3/16	3/16	1/16	0
Sept. 1	1/16	1/16	3/16	3/16	1/16	1/16
Sept. 3	1/16	1/16	1/16	1/16	1/16	0
Sept. 10	3/16	1/2	11/16	3/4	1/2	1/4
Sept. 15	1/16	1/2	1/16	7/8	1/16	1/16
Oct. 1	1/16	3/4	1 1/16	1 1/8	1 1/16	1/16
Oct. 8	3/8	1/8	1 1/16	1 1/16	1 1/16	1/16

The anchorage bars began to lift with the traveler at panel 4. As noted in a previous article (Jan. 7, 1915), the diagonal tension members and top-chord eye-bars of the anchor arm had been slotted in order to facilitate erection while the truss was in its cambered position. As the load was added to the cantilever arm, these tension members came to a bearing by degrees until the whole anchor arm was self-supporting.

When the erection traveler is removed, there will be a positive reaction at the anchor pier of about 200,000 lb.

*Assistant to Chief Engineer, Board of Engineers, Quebec Bridge, New Birks Building, Montreal.

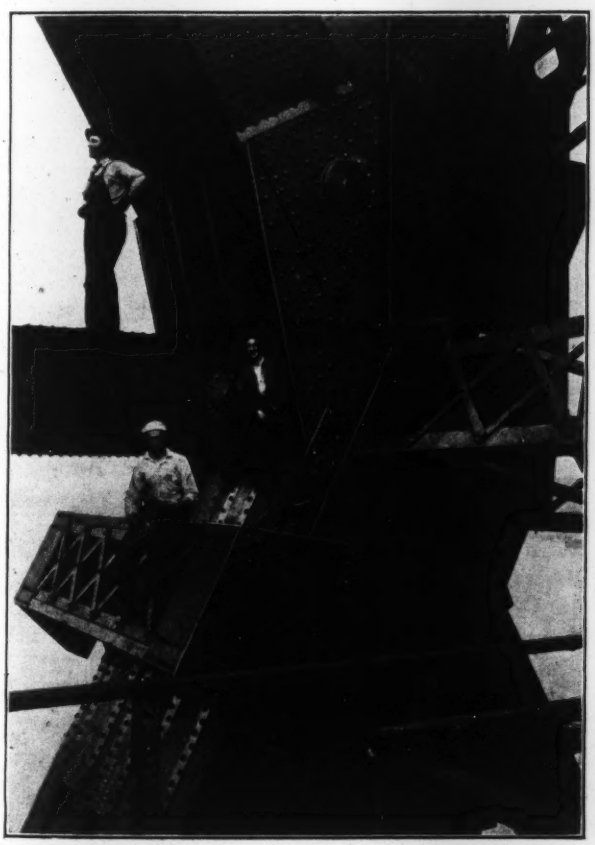
In order to put a tension in the anchor bars, the falsework bent under panel 2, next the anchor arm, will be left in and jacked up under a load of 600,000 lb. until the suspended span is swung.

The last work performed this season on the bridge was to erect and place the eye-bar suspenders for the suspended span, and to place in position the heavy jacking girders—on the extreme end of the cantilever arm—which will be used for supporting the jacks used in hoisting the suspended span after this is floated into position. As the eye-bars supporting the suspended span in place will be subject to constant movement, due to expansion and contraction of the bridge, the pinholes at the top and bottom sections have been fitted with phosphor-bronze bushings.

At the present time all falsework under the anchor arm has been removed, with the exception of the bent under panel 2, as mentioned above; and the erection traveler is being dismantled and taken down. This traveler will be used next season for the erection of the suspended span at Sillery Cove, some three miles down the river from the bridge, where falsework is being built close along shore as staging for construction of the span.

Progress of the South Cantilever

The inner falsework used on the north side to support the floor of the north anchor arm was last spring taken down and shipped to the south side and used in a similar position. As the shore line on this side is at a different elevation than on the north side, this difference in elevation was taken care of in the design of the falsework by leaving out or adding parts.



MIDDLE INTERSECTION OF FIRST MAIN DIAGONAL TO NORTH SHORE CANTILEVER ARM

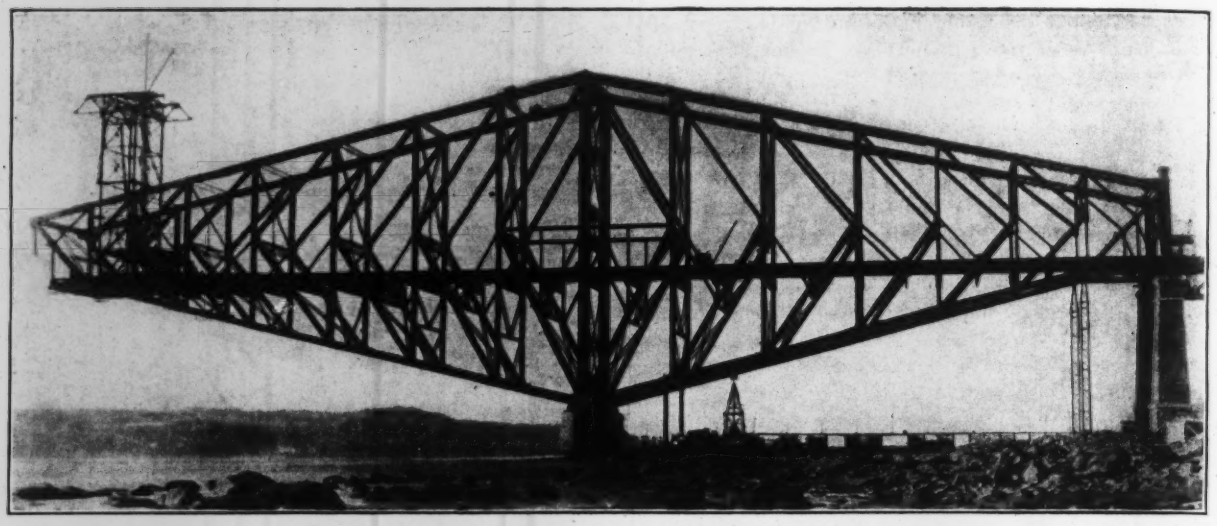
HOW THE STRAIGHT-LINE CANTILEVERS WILL APPEAR FROM RIVER. (NORTH CANTILEVER, OCT. 25, 1915)

From the experience gained on the north side, very much better time was made on the erection of the south anchor arm, a saving of some six weeks being made. As on the north side, this work went together without any trouble, and the contractor was able to improve on the program laid out in the spring. Owing to the extra progress made on the south side this year, the contractor will have at least six weeks' advantage next spring and can start immediately on the erection of the cantilever arm. According to the present program, the south cantilever arm should be completed some time in September, 1916, at which time the bridge will be ready for coupling up the suspended span. As work up to now has gone ahead of program schedule, the bridge is likely to be ready for this operation at the time fixed.

Work on Suspended Span

The inside falsework under the floor of the south anchor arm has been removed and will be sent back to the shop to be remodeled for use as falsework for the erection of the suspended span. The concrete piers upon which this falsework will rest have already been put in place between low and high water at Sillery Cove, and early next spring a start will be made on the erection of the span.

The construction of the scows necessary for supporting the span during floating operations will also be started this winter. Six scows will be required, each 160x32 ft., with a maximum draft of 8 ft. under working conditions. The span, as floated into position, will be 640 ft. long, 88 ft. wide and 110 ft. high at the center, and will weigh approximately 4,700 tons. Steel links supported from the jacking girders at the end of the cantilever arms will be attached to the end of the suspended span, and with the falling tide the scows will be moved from under the bridge. During lifting operations all traffic will necessarily have

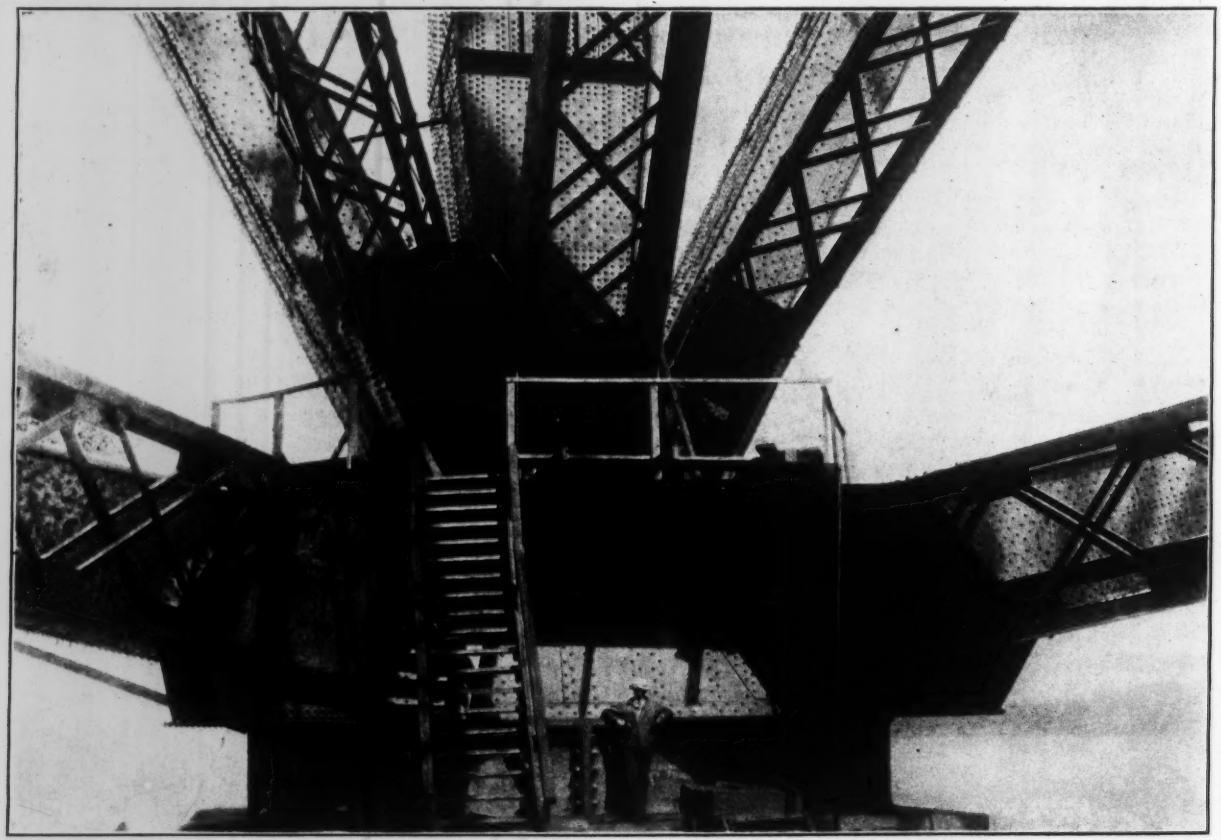


NORTH CANTILEVER OF THE QUEBEC BRIDGE ON DEC. 2, 1915

Falsework bent at panel-point 2 left in place to jack up for securing anchorage tension; erection traveler being dismantled

to be stopped in the river. If everything works according to the schedule, lifting operations should not take more than 24 hr. from the time the suspended span is hung to the end of the cantilever arm. By next fall it should be possible, after many years' effort, to have the two sides of the St. Lawrence River linked together at this point, finally carrying the great government railway, the National Transcontinental, across the St. Lawrence to its connections reaching the Atlantic coast.

The St. Lawrence Bridge Co., of Montreal, is the contractor for this work. Phelps Johnson is President; G. H. Duggan, Chief Engineer; George F. Porter, Engineer of Construction; W. B. Fortune, Superintendent of Erection; S. P. Mitchell, Consulting Engineer of Erection. The work is under the supervision of the Board of Engineers, Quebec Bridge, Department of Railways and Canals, Canada. The board consists of C. C. Schneider, Ralph Modjeski and C. N. Monsarrat, Chairman.



MAIN SHOE OF QUEBEC BRIDGE, WHERE TEN MEMBERS COME TOGETHER

Pick Rational Plant Units for Depreciation Accounting

By Robert G. Klotz*

The determination of what character of expenditure shall constitute in depreciation accounting a "renewal," and shall therefore be provided for in the depreciation account, and what shall constitute a "repair," and therefore be paid for out of expenses and regarded as maintenance, is a problem much encountered and the source of a great deal of misunderstanding between engineers and accountants, particularly in railway work. The application of a little commonsense will usually overcome the difficulties and result in a workable, if perhaps not a perfect, arrangement.

This question has been made a still more difficult one of late because of the detailed character of the appraisals prepared by engineers, voluntarily or to comply with the requirements of official bodies, bringing out the value of every possible detail. Without due regard to the true meaning of the actual depreciation in the physical condition of the property as shown in such reports and to the underlying purposes of depreciation accounting, accountants have in some cases attempted to employ the engineer's detailed valuation as a basis on which to form their depreciation accounts. Such procedure, while it may not result in any insurmountable difficulties, inevitably involves the accountant in a most intricate tangle of detailed accounts and in frequent misunderstandings with the engineer, whose purpose in accurately detailing his valuation was not to imply the need of writing up depreciation on each individual item entering into the construction of the property, though to the accountant it may so appear.

The real purpose of the depreciation account is to spread over the years of useful life the payment of heavy anticipated charges for the replacement of parts of the property when they can no longer be kept in operating condition by reasonable expenditures for repairs, where such charges, if met in one payment, would make the total expenditure for the year in which they are incurred disproportionately large.

Agree on Plant Units for Accounting

The selection of the proper units of plant upon which to base the charges should be made by agreement between the engineer and the accountant, or by some one person conversant with the requirements of both branches of the work. In this way the engineer can be certain that the depreciation fund will at the proper time be reasonably sufficient to cover the charges which will have to be made against it, and the accountant may be assured that he will not be required to carry an unnecessarily heavy burden of detailed accounts which might include items properly to be classed as repairs. In determining upon proper units there is ample opportunity for disagreement, and here the employment of sound commonsense and resort to reasonable compromise will be needed.

For true economy and for convenience the units used should be as large as is compatible with the equalization of the annual maintenance charges, and should be such units, or such parts of larger units, as it is fairly certain will have to be renewed as such.

In writing up depreciation on a generating station it would naturally occur to one that each generating unit, comprising prime mover, generator and appurtenances, might well be taken as the unit for depreciation, and an annual charge written up sufficient to provide for its replacement on the expiration of its estimated life. Here the engineer may, however, show that a certain appreciable portion of the unit may require replacement before the end of the assumed life; for example, that the insulation of the armature coils would be very likely to fail and that the coils would have to be replaced while the remainder of the unit was still in good operating condition.

Should it be necessary to renew the entire set of coils at one operation, an expense would be involved which might tend to increase unduly the expenditures for that particular year, in which case the armature coils or winding should be covered by a separate depreciation charge. Were it possible to replace individual coils from time to time, this would incidentally divide the unserviceable period of the unit into a number of short intervals rather than one long interval. Then the cost of the replacement might well be distributed over two or more years, in which case the expenditure in each year would not be sufficiently great to unbalance the annual maintenance charges, and the depreciation could be written up on the generating unit as a whole, instead of on two or more parts thereof.

Avoid Excessive Division of Plant

It is evident that whether the units assumed be large or small, the total cost of maintaining a unit, that is, annual repair and depreciation charges combined, should be the same. Where, however, the larger unit is used, as in the second situation cited, and it is possible to renew portions of a unit under the heading of repairs, depreciation is written up on each such portion only once during the life of the unit, although each portion may be replaced several times during that period. In this way excessive division of the depreciation account is avoided, and the account is at the same time kept at a minimum—a matter of some importance when it is considered that the amounts charged to the depreciation account must be regarded as practically lying idle.

Were it possible so to regulate the total annual cost of repairs and renewals in a plant that the sum involved would vary but slightly from year to year, or perhaps rise inversely with the decrease of value in some cases, a depreciation account would be obviously both unnecessary and undesirable; and if engineers and accountants will come together and discuss the question on a rational basis, they can in many cases closely approach such a desirable state of affairs. By extending the life of a certain unit and reducing the extent and cost of the repairs made during a certain year, with the knowledge that it will be removed from service during the year following, an amount may often be saved sufficient to provide for the renewal, covered by current funds, of another similar unit, the saving on repairs on the new unit during its first year of use providing sufficient leeway in the current funds for that year to cover the cost of replacing the unit previously allowed to deteriorate to the end of its usefulness.

To illustrate: If a certain length of track of a railway, all of which is subject to equal wear, has an assumed life of 20 yr., a sufficient amount may be charged to the depreciation account each year so that at the end of the 20 yr. the funds on hand will be sufficient to provide for its replacement, the cost of which will thereby be distributed

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over the intervening years. On the other hand, it may develop that the cost of this particular replacement could be divided into, say, six annual charges without increasing unduly the total annual cost. By dividing the length of track into six approximately equal parts and renewing the rail of one such part during each of the six years beginning and ending with the eighteenth and twenty-third years of life, the replacement can be made as a repair from the accountants' point of view and the necessity of maintaining a nonproductive depreciation fund avoided. In such a case the rails removed during the earlier years would be used in extending the life of the sections renewed later, resulting in a better average condition of the length of track during the final years of life than would be the case were the replacement to be made as a whole and deferred until obligatory.

Accountants' Reconciliations

I have come into contact with cases where the accounting representatives of a Public Utilities Commission attempted to reconcile the reports of the engineers as to the miles of track of a railway company renewed during a certain year with the total amount of rail purchased by the company during that year. Insistence on this point would mean that the replacement of any piece of rail, however small, would constitute a "renewal," and that depreciation accounting, if it were duly carried out, would have to be based on a unit not only small, but variable and of indeterminate life. Even approximate determination of the life of a certain piece of rail in a length of track would be impracticable. The engineer may be able to decide with some degree of accuracy as to the probable life of the rails in a given length of track regarded as a whole. Any piece of the rail may, however, break and require replacement in a short time, while another piece may not only remain sound until such time as the dictates of safety require its removal because of direct wear, but may give a long term of further service in a shop, yard or sidetrack.

With such items as buildings and rolling-stock it would of course be impracticable to attempt to divide the cost of replacement in such a manner as to provide for its distribution under repair charges, and in that case it behooves the engineer and accountant to "get together" and to decide on the unit which will prove most satisfactory from their combined point of view.

There was brought to my attention quite recently a case in which the engineers and accountants of a railway company, being unable to come to any mutual agreement as to the proper line of division between "repairs" and "renewals," referred the question to their counsel for decision. As far as I have been able to determine, the decision has not as yet been rendered; but as it is quite likely to be one based on purely legal opinion, devoid of knowledge of either engineering or accounting technicalities, it may well be assumed that the engineers and accountants will both very likely find themselves out of the frying-pan and in the fire when the decision appears.

New Topographic Survey Methods for Rapid Work

By James H. Bonner* and Frank E. Bonner†

SYNOPSIS—Radical changes in time-honored methods of taking topography of timber lands are described—use of an improved Abney level, or clinometer, and a special tape, which employed together give directly the horizontal distance from a slope measurement. The method is described in detail.

The last few years have witnessed a remarkable change in the logging industry of the Pacific Northwest. It is only a few years ago that a man with a quarter-section of timber land, a team of horses and limited capital could engage in the logging business. Times have changed; the close-in timber has been logged; keen competition has reduced prices; and the future logging operations of the large companies are planned out on a comprehensive scale for years to come. The old "tote" road is being supplanted by the logging railroad. The passing of the small-mill man is marked by the advent of the logging engineer, a profession unknown a few years back. Practically all of the companies doing railroad logging in the Northwest now employ more or less continuously an engineer, who plans and directs the entire field operations.

Time was when a logging railroad followed the general contour of the ground, with narrow roadbeds, steep grades and insufficient drainage. Modern business efficiency no longer permits such methods; the logging companies are now doing on a small scale what our transcontinental railroads are undertaking—eliminating sharp curves and reducing grades on the main lines.

Such construction increases first cost, but the logging engineer no longer plans on tearing up the steel in a few years when the area has been logged. The subgrade is so constructed that the owner of timber beyond can afford to buy the road and haul his logs out over it in preference to building another railroad. Also in special cases, owing to the present-day demand for logged-over lands for farming purposes, there is a chance of the road being used as a common carrier in years to come.

Accurate Topographic Map Essential

The logging engineer's first demand in taking over a new job is for an accurate topographic map, not only of the area that his company intends to log but of all the adjacent territory. There is no need to dwell on the necessity for an accurate contour map. It is the working plan of the logging engineer, on which he makes the locations for railroads, spurs, flumes, chutes, roads, camps and cable systems.

The cruiser's maps giving general locations of streams and ridges with occasional elevations taken with an aneroid barometer served their purpose in their day, but the modern logging business requires more detail. Topog-

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raphy must be accurately mapped, reports must give a careful classification of all timber, viz., species, standing, dead and down, timber suitable for piling, poles and ties, with an estimate of probable defects; also soil classification, conditions of undergrowth and mineral indications.

Topographic surveys for logging operations are made in various ways. The method to be used is determined by the lay of the land and the density of the forest growth. Until a year or two ago the custom was to make all such surveys by the aneroid method, which is simply a refinement of the methods formerly used by timber cruisers. Elevations are determined by primary leveling along certain lines selected as base lines. If the area has been previously covered by the survey of the United States General Land Office, it is customary to establish the base along the section lines; while in unsurveyed territory irregular traverses are run along roads, trails or ridges, the work usually being done by transit and stadia and the elevations computed at fixed intervals. From these base lines parallel "strips" are run.

The strip crew usually consists of a compassman and an estimator. The compassman runs a rough line with a box compass and paces the distances. He carries an aneroid barometer which he sets each morning from the reading of a camp barograph. He sketches the contours each side of the line run. It is necessary to record the time at which the aneroid readings are taken in order to make the corrections for the fluctuations of the instrument during the day.

The aneroid, while a handy instrument for taking rough elevations, is too eccentric to be relied on to do accurate work (as any engineer is aware who has used it in mountainous regions subject to frequent and sudden atmospheric changes). As the working aneroid goes through the changing atmosphere conditions during the day, it is necessary to correct the readings in proportion to the fluctuations of the camp barograph.

After the day's fieldwork is completed and the aneroid readings corrected, the difficult work of adjusting contours

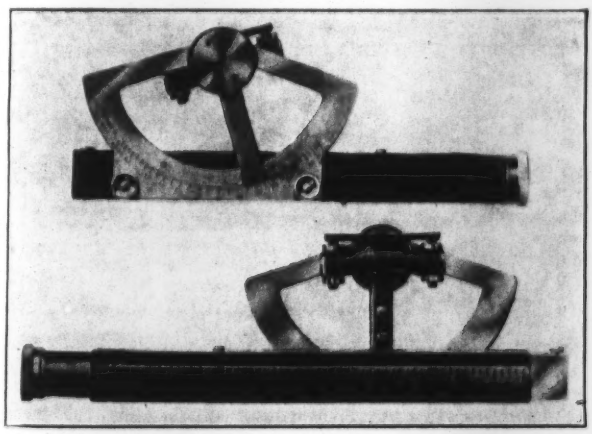


FIG. 1. IMPROVED ABNEY LEVEL OR CLINOMETER

on the finished map still remains. If the topographer has been conscientious and is skilled in his work, he has gathered a wealth of topographic detail which is all to be lost and wasted when his contours are put through the "juggling" process. The very lack of preciseness and the knowledge that the map must be adjusted greatly discourages careful and conscientious sketching in the field.

It is very evident that the map cannot be accurate. All topography is approximately correct, but it would hardly answer as a map on which to make the paper location of a railroad or a flume. The cost of making surveys using the method outlined is from 5c. to 35c. per acre.

Such was the condition when the engineers of District No. 1 of the United States Forest Service commenced their series of experiments to develop a more efficient method of topographic surveying. Ordinary topographic-survey practice was not applicable. Heavy timber and dense

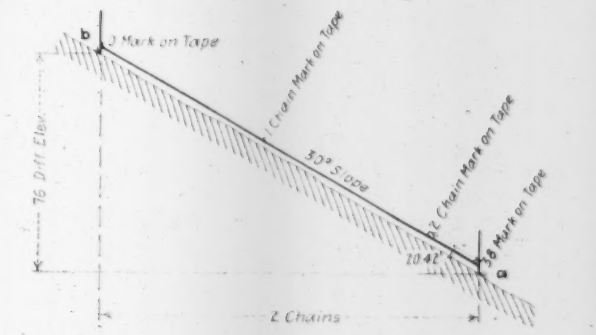


FIG. 2. ILLUSTRATING USE OF SPECIAL SLOPE TAPE

brush and undergrowth prevented efficient use of the plane-table method, as it was necessary to get beneath the cover of the forest in order to get the topographic detail required. Also the operations must necessarily be combined and coördinated with the timber estimating and appraisal work and could not therefore follow the methods that would be adopted for the production of a topographic map alone. Low cost was an important consideration.

The first tests were made with the ordinary Abney level, or clinometer, familiar to all engineers. While the results were satisfactory, it was a cumbersome method, in which the field men were compelled to constantly refer to reduction tables in order to ascertain their true horizontal position. The instruments as then made were not constructed for such work, and it was almost impossible to keep them in adjustment. By constantly experimenting and making improvements a new Abney level was developed that is now manufactured [by Keuffel & Esser, New York] expressly for work of this character. Simplified methods introducing greater speed and accuracy were worked out.

New Abney Level and Its Use

The improved Abney level, illustrated in Fig. 1, is only slightly larger than the old Abney, and its cost is about the same. The arc is made larger and graduated to read directly the difference in elevation per chain (66 ft.) of horizontal distance. A prism has been substituted for the german silver reflector, and the bubble tube is made longer, with adjustment devices similar to those on the bubble tube of a transit. The bubble-tube support is attached rigidly to the indicator arm of the arc, thus eliminating the inevitable lost motion between these two parts existing in all old-style Abneys.

The bubble tube is arranged to allow adjustment to eliminate refraction, which made the old-style instruments so unreliable in measuring steep slopes. A semicircular lens placed in the fore end of the eye-piece tube, as in the Locke level, magnifies the movement of the bubble so as to greatly facilitate accurate sighting. The various

changes introduced have so improved the instrument and the results of its intelligent operation have proved so surprising that the new Abney will doubtless meet with favor among the entire engineering profession, and its use in fieldwork of all kinds will probably be greatly extended.

Aside from the special graduation shown in Fig. 1, which is applicable to all topographic work, plates with the usual degree and per cent. graduations are furnished, and also a special graduation to give directly horizontal distance from slope chaining. This graduation has found much popularity in cadastral surveys.

Topographic Surveys of Timber Lands

There was considerable objection at first to the Abney method on account of the necessity of taping the slope distances, but two years of trial have proved that this work can be done with practically the same convenience and time as pacing.

The unit of measurement adopted was the chain of 66 ft., for the reason that a large part of the work is the retracement of land-survey lines recorded in that unit,

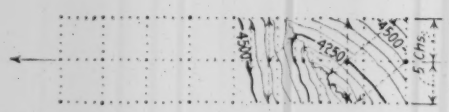


FIG. 3. LOCATING CONTOURS BY THE ABNEY METHOD

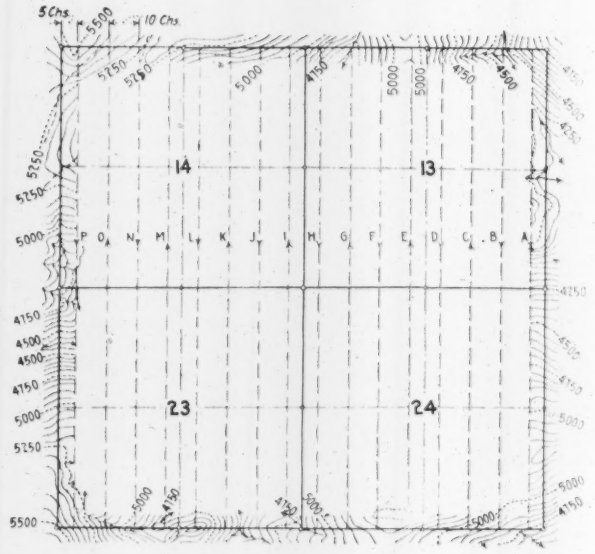
and also for convenience in timber estimating. Obviously a definite horizontal unit is necessary for operation without the use of tables and after thorough trial it was found that a distance of 2 chains was most convenient for rough country covered with heavy timber. A special 2½-chain tape has therefore been designed for use with the Abney method, the tape proper consisting of 2 chain lengths divided into links, with an additional half-chain "trailer." The trailer is graduated in order to allow the proper slope measurement for the excess in length of the hypotenuse of a triangle over a base which is 2 chains.

For example, suppose the crew to be ascending a slope, as indicated in Fig. 2. The topographer reads from the arc that the rise per chain on that slope is 38 ft. The rear chainman (usually the timber estimator) then looks for the 38 etched beyond the 2-chain tag on the tape, which in this case would be 20.42 ft. beyond. By stretching the tape tight, the point *b* is then located and is known to be 2 chains distant from, and 2x38 ft. higher than, *a*. The topographer then sketches his contours, taking side-shots in order to locate the distance between contours and "sidewalking" if necessary.

A strip crew of two men will cover about two miles of line a day. Frequently the brush and undergrowth are so thick that sights two chains long are impossible, requiring the use of a one-chain distance or sighting by sound. Remarkably good closures have been obtained in many cases employing the latter method. In dense and dark forests it is usually necessary for the estimator to carry a small mirror in order that the topographer may sight on the flash. In this way sights are obtained through brush that seems almost impenetrable to the eye.

The general method followed is similar to that used in the aneroid method. Primary control fixing the geographic position of the area both horizontally and vertically is obtained by a precise connection to adjacent primary stations of the United States Geological Survey. Secondary control lines are run around a block of sections, or topographic unit, stakes or posters on trees being left at periodic intervals, from which the strip surveys may be begun or closed.

The Abney instrument is used for running the control lines as well as for the strips. It has been found to be about as accurate as direct leveling and much faster. The



FIGS. 4 AND 5. STEPS IN THE TOPOGRAPHIC SURVEY OF FOREST LAND BY THE ABNEY METHOD

work on control lines is of course much more carefully executed than on the strip lines, two Abneys being used.

The control system is so planned that the strip surveys will not be run more than two miles without closing upon a control line. The strip surveys are run along parallel lines usually ⅛ or ¼ mi. apart, depending upon the density of the timber and the degree of detail required.

It is customary to combine the mapping with either the timber-estimating or the soil-classification work, thus making one survey serve two purposes and reducing the

cost. The crew consists of the topographer, who keeps his direction with a staff compass, operates the Abney and sketches the topography; and the estimator, who "snubs" the rear end of the tape, making the proper allowance for slope, and estimates the timber on a strip 33 ft. wide on each side of the surveyed line. Experience has shown that the time lost by the estimator in holding the rear end of the tape is practically negligible.

The strips are usually 10 chains wide, the line being run through the center of the strip, and the topography sketched for 5 chains on either side. The method is not a system of determining the elevation of points along a profile line and interpolating the intermediate contours, such as the aneroid method is, but a number of points are located by taking side shots at right angles to the general direction of the contours, as indicated by the dotted lines and arrows on Fig. 3. Although the skeleton of the topography is thus accurately obtained mechanically, the sketching or filling in of the topographic detail depends entirely upon the skill and judgment of the topographer, which may be attained only through considerable experience and some amount of natural talent, often called "topographic sense."

As the price of stumpage increases, the accuracy and detail demanded on topographic maps increase. Some of the larger lumber companies now have their strips run at 2½-chain intervals, estimating each tree separately and trusting nothing to averages.

Accuracy of Abney Method—Errors of Closure

The results obtained by those who have been working with the Abney method outlined are truly marvelous. On one project comprising 54,000 acres in the Cœur d'Alene National Forest in northern Idaho, completed during the 1915 season, 64 mi. of control line was run with an average closure in elevation of one foot to the mile.

The area mapped was very rough and mountainous, and the timber cover consisted largely of heavy stands of Idaho white pine accompanied by heavy undergrowth and dense brush. The greater part of the area had been previously covered by both the General Land Office surveys and by the Geological Survey small-scale topographic quadrangles, so that no primary control had to be executed. On this same project over 500 mi. of strip surveys were run with an average closure in elevation of 10 ft. to the mile. The minimum error on such lines was one foot to the mile, while the maximum allowed was 30 ft.

The errors of closure for alignment and distance on the strip surveys each averaged ½ chain to the mile. Mapping was done on a 4 in. = 1 mi. scale, using a 50-ft. contour interval. It is readily apparent, therefore, that the errors of closure resulting can be readily adjusted so as to be almost negligible.

The cost of the work, including the control, mapping, timber estimating, etc., was approximately 12c. an acre.

FIGS. 6 AND 7. TYPICAL IDAHO FOREST SURVEYED BY ABNEY METHOD

The results of this project furnish an average example of what is being accomplished on a large amount in similar work. In less rugged and more open timber, as in the yellow- and lodgepole-pine stands, for example, closing errors and costs are more favorable.

Testing Concrete in Roads—The United States Office of Public Roads and Rural Engineering has begun a series of tests at Arlington, Va., to determine the proper thickness of the concrete base for roads of different types when subjected to loads of different magnitudes and when placed on various kinds of sub-base. In view of the increasing intensity of wheel loads to which roads are now being subjected, the solution of this problem is becoming of great economic importance. Steps have been taken to mold slabs of varying thicknesses on several kinds of sub-base and test them to destruction, the load being applied through various kinds of paving surface. Concrete roads are now being built throughout the country with numerous kinds of aggregate and under various construction methods. There is a noticeable difference in the amount of wear suffered by these surfaces, and it is a much-mooted question as to which kind of aggregate will prove most economical under particular conditions of traffic and location of source of supply. In order to gain information of value in this connection, an instrument has been devised and constructed in the concrete laboratory at Arlington, Va., for the purpose of measuring the amount of wear of concrete roads of different types. Periodical wear measurements will be taken of concrete-road surfaces on which the traffic conditions are known. It has long been known that concrete is not elastic, but attains permanent deformation under working loads. During the course of a series of tests on reinforced-concrete slabs recently made by the Office of Public Roads and Rural Engineering, and now being continued, it was very forcibly brought to the notice of the observers that the concrete continued to deform slowly as long as the load was applied. This fact is significant, as it helps to explain certain stress conditions in concrete roads and bridges when subjected to temperature changes. In order to study this phenomenon more fully, a series of tests has been begun to determine the "flow" of concrete and reinforced concrete when subjected to direct compression and to bending stresses. Working conditions will be maintained on the specimens as long as they exhibit "flow," and finally the specimens will be broken in comparison with like specimens that have not been previously loaded. From the Departmental Circular of the United States Department of Agriculture.

Field and Office

Cleaning Hollow Piles With a 10-In. Orangepeel

By G. L. Knight*

In building a heavy steel platform foundation for turbine No. 3 in the Gold St. generating station of the Edison Electric Illuminating Co. of Brooklyn, N. Y., it was necessary to put down a number of 12-in. tubular-steel piles, which later were filled with concrete. The piles were dredged out by a method employed only infrequently—by the use of 10-in. orangepeel buckets. The piles could not be blown out with air, owing to a buried timber cribwork dock and many boulders through which the piles were driven. The use of water had the added objection of possibly undermining the adjoining foundation of No. 2 turbine. The work was done in cramped quarters. Before driving could commence, an existing mass-concrete foundation, which occupied the site, had to be cut out.

FIG. 1. THE DWARF BUCKET USED TO DREDGE OUT PILES, WITH SAMPLES OF THE SPOIL

The piles used were cylindrical sectional piles of 12-in. inside diameter, driven in sections of convenient length, the joint being made by means of a sleeve 12 in. long and of the same outside diameter as the inside of the pipe. The sleeves were held in place by centerpunching the pile sections 6 in. on each side of the joint. The material was excavated from inside as the pile was driven. The first sections used were 18 ft.

On account of the proximity of No. 2 foundation, which was not piled, but whose bottom rested directly on the soil, 12 ft. below the basement floor, it was not deemed advisable or safe to jet the piles. The method adopted was to drive as far as possible and then excavate with a 10-in. Hayward orangepeel bucket inside the 12-in. steel pile.

The piles drove anywhere from 5 to 10 ft. in the first driving. The instructions given the inspectors were to drive until the pile came to refusal (¼ in. in three 5-ft. drops of the hammer) or until they heard the ring of a boulder or got the spring of timber.

At the end of the first driving the clearing crew started in, using an auger, a chisel bar and the bucket. The auger proved of little use as there were too many small stones and other obstructions to permit its turning, although it was used whenever the chisel bar showed that there was a section of sand or soil fairly free from stone. The procedure throughout the work after each driving was to use the chisel bar to loosen up the material inside the pile and then with the bucket remove the material to within a foot of the bottom of the pile.

Piles Encountered Many Obstructions

Over part of the site of this foundation there had apparently been an old crib dock or wharf whose bottom rested some 25 ft. below datum (basement floor is datum), and this accounted for the many obstructions encountered. These were principally boulders, rough stone and the timber itself, many pieces of which lay horizontally in the path of the piles and were cut through and a section brought up by the bucket. Fig. 2 shows a few typical prizes recovered.

As is usually the case with such jobs, the progress was very slow and unsatisfactory at the start of the work, but as soon as the crews got used to their duties the

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work went on with remarkable speed considering the difficulties encountered. There were two driving crews (one day and one night), and three excavating crews (two day and one night). Three buckets were used, so as to be sure to have one spare at all times.

At the north end of the excavation a section of concrete was encountered, which had once been used as a sump and which lay directly in the path of the three piles at the north end. As the top of this concrete was some 12 ft. below the surface, it was impracticable to excavate it in the open, and it was determined to drill three holes in the concrete to admit of the piles being placed in their proper positions. A well drill was secured and placed in the ways of the piledriver, and the three holes—15 in. in diameter and about 5 ft. deep—were cut through in four days. The three piles were then inserted in the holes and driven to place.

The depth set for the bottom of the piles, after careful examination of the soil excavated, was 10 ft. below the bottom of the condensing tunnel, or 34 ft. below datum.

In some cases where the pile encountered a boulder but glanced off, notably in the case of piles 8 and 11, the bucket was blocked after one driving, although the driving indicated nothing unusual. Where the pile had been driven only a few feet, it was pulled up and replaced with a slight change in location, the work proceeding then very cautiously, using a smaller core drill inside the pile to break through the boulder in case it was hit again. Not only were the bottoms of the piles damaged, but in some cases the second section was telescoped with the first. In another case the sleeve slipped and the upper, or head, section was driven eccentric to the bottom section for a distance of some 2 ft. before it was discovered.

As the piles were laid out as close together as practicable, it was not possible to drive other piles to make up for the loss of these. Each case had to receive careful consideration on its merits. Sometimes the pile was pulled, and where this was impossible it was filled up and the piles adjacent driven to greater depth. In still other cases, where the break was near the surface, a sheeted pit was made around the pile, the material excavated in the open until the joint was uncovered, the damaged steel cut off and a new section put on. In all, 23 piles were driven in one month, working 24 hr. per day. Subsequently, load tests of 40 tons were made on several of the piles.

Erecting a Bascule Bridge

Demands of navigation quite frequently make it necessary to erect bascule bridges in an open position. This requirement brings up special problems in so locating and adjusting the structure that it will operate smoothly and have even bearing when closed. The following notes give some of these problems and their solution in the case of a bascule span of about 175 ft. clear waterway.

The girders were fabricated in three sections, *A*, *B* and *C* of Fig. 1, with field splices along lines *X Y* and *Y Z*. The first steel erected was the rack and its support, which provided means for staying the rocker section *B* until the temporary bottom lateral system could be placed. The rocker section was placed in the extreme open position for the span and held there by the erection strut, which engaged one of the slots in the tread at its top end and was wedged against a tooth of the track plate at the bottom (Fig. 2).

It was found that in fabricating the sections *A* and *B*, which have a heavy flange along one side and nothing but the web plate at the other, the riveting of the flange had caused that edge of the section and web plate to stretch and the other, unstiffened, edge to buckle. When section *A* was put in place, it was found that the long splice *X Y* came together at the ends while the plates were still considerably separated at the center. Yokes and clamps were of very little assistance in getting the splice together, the most effective method being to use a dozen or more drift-pins in the splice plates at one time. In one case it was impossible to get fair holes by ¼ in., and the $1\frac{3}{16}$-in. holes were therefore reamed to $1\frac{1}{16}$ in. and 1-in. rivets driven.

To align the girders so that the rolling segment would be parallel to the bridge center line, a transit was set

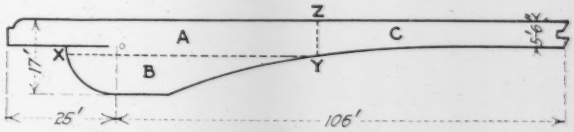


FIG.1 CANTILEVER GIRDER FOR ONE LEAF

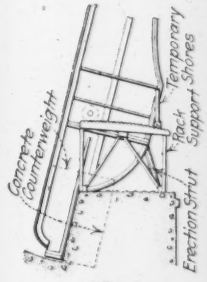


FIG. 2

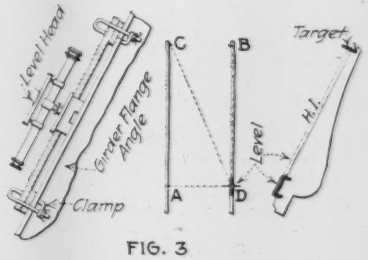


FIG. 3

FIGS. 1 TO 3. DETAILS OF BASCULE LEAF BRIDGE

Fig. 1—Bascule girder made in three erection pieces. Fig. 2—Heel portion of bascule leaf as erected. Fig. 3—Alignment sighting to detect twist of girders

up on the track plate in a position offset from the girder center line because of the erection strut's interference. A foresight with the same offset fixed the plane parallel to which the rolling segment was to be adjusted. By taking a position with the transit on the girder center line across the channel, it was determined how much the tops of the girders were away from the vertical. The necessary adjustments to bring the tops to correct position were made by tightening with steamboat ratchets the temporary wire-rope diagonals of the lateral system. To set the girders so that they would be in the same position of roll, a steel rod gage or standard was used, with which the distance from the center of bearing (closed position) on the track plate to the same point on the rolling segment could be measured. Adjustment could be had by the wedges at the foot of the erection strut.

Certain blocking against the counterweight box was inadvertently removed just before concreting was commenced. The result of this mishap was to allow one girder to roll back to a more nearly erect position, enough to lift it off the erection strut ⅛ in. When a check was made to find the effect, it was seen that one girder was ahead of the other at the top, which was determined by the following method:

The legs were removed from a Y-level, which was then mounted on a board (Fig. 3). This board was clamped to the cover plates of the girders so that the axis of ro-

tation of the telescope was normal to the bridge floor. The H.I. was taken as the distance from the back of the flange angles to the line of sight of the telescope. The telescope was then adjusted by the leveling screws until it was made to revolve in a plane parallel to the three points *D*, *B* and *A* (Fig. 3) at a distance equal to the H.I. above them. A reading was then taken to the back of the flange angle at *C*, which gave the error in its position. Self-reading targets were used at *C* and *B*, bolted to the girders.

The concrete had set by the time the error was discovered, and it held the two girders firmly in position relative to each other. It was necessary that the outer ends of the girders be in correct position, or else the bridge would not lock properly when closed. As no adjustment was possible at the lower end, the upper ends had to be rotated slightly about the splice *ZY* (Fig. 1), the holes in which were then reamed. Half of the error was taken up in each girder.

On the opposite side of the river the only trouble encountered was in the position of the track girders. They were carried by buttresses that were concreted to a point slightly below the position of the bottom flange of the track girders before the latter were placed. The track plates were set accurately to level and to line and station. The concreting was then finished. Afterward it was found that the pressure of the concrete had in some way become unbalanced on the two sides of the girders, and one of them had moved, making the distance between the track plates too great by ½ in. In this case it was possible to take up part of the discrepancy in the setting of the rolling girders and part by increasing the spacing of the girders, but the accident suggested the desirability of providing temporary sway-bracing between track girders in such cases.

Occasions for the use of level or transit with legs removed, as in the case above, come up very frequently in the engineering work on bridge erection, and the instrument man should remember that the legs are by no means an essential part of his apparatus. Many times the use of the head alone will make possible a set-up which would otherwise be difficult or impracticable.

Overdriven Piles and Poor Subaqueous Concrete

By R. W. Stewart*

The bed of the Los Angeles River is composed of sand and gravel nearly free from clay or silt. On account of the excellence of this material large quantities are hauled out for construction work. The rate at which the gravel is replenished by floods carrying down a new supply from above is much slower than the rate of removal. As a result the channel through the city has lowered 10 to 20 ft. and it has become necessary to underpin or reconstruct the piers of all the older bridges. This work has uncovered some engineering data of interest.

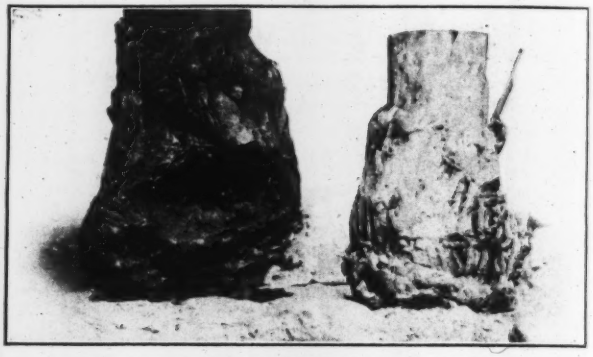
Several of the bridges at sites where groundwater interfered with excavation to suitable depth had pile foundations under concrete piers. It was specified in all cases that these piles be driven to refusal, and evidently there was no laxity in enforcing the specification.

*Engineer of Bridges, City of Los Angeles, Calif.

Of 280 piles which have been excavated, 130, or 47% were either broomed or telescoped in driving. By a broomed pile is meant one which is split and expanded into a mass of fibers at the bottom, thus causing it to resemble a broom. Some of the telescoped piles had a neatly formed telescopic joint which resembled a pipe coupling. The view herewith shows the lower ends of some broomed piles.

An experienced piledriver foreman can tell when a pile becomes broomed in driving by observing that it rises slightly immediately after the blow of the hammer. It is probably needless to say that a broomed pile will settle under load and is of little value. Care should be exercised to stop driving before rupture of the pile begins.

The concrete in the lower portions of some of the piers had been deposited under water. This concrete was in



LOWER ENDS OF TWO BROOMED PILES

such condition that a man wielding a piece of wooden two-by-four by hand could punch it to pieces. The fact was well illustrated that in depositing concrete under water it is necessary to use a device to prevent free concrete from sinking through the water.

In reconstructing these piers it was desired entirely to avoid placing concrete in water. Difficulty was encountered, due to the streams which would flow in through the sides of the foundation pits and surge up vertically at points in the floor of the pit. The only satisfactory manner of placing the concrete was found to be by building a canvas boat which was enough smaller than the interior of the pit sheathing to allow clearance for the flow of water and the suction pipes of the pumps around it. The water was then pumped down and the boat, while resting on the bottom, was filled with concrete.

Practice in Sewer Construction at St. Paul, Minn., is described in the recently issued annual report of the Commissioner of Public Works as follows: All sewers of 36-in. diameter and under have been constructed of vitrified clay pipe. For large sewers in open-cut work the practice has been adopted of giving the contractor the option of building the sewers of monolith reinforced concrete or of vitrified segmental sewer blocks. The contract plans are drawn showing the details of construction to be used in either case. The specifications require that in wet or soft ground the segmental blocks shall have a concrete backing and in such cases as have occurred, where these conditions were met with, it has proved more economical to use the reinforced-concrete type of construction. All concrete sewers are provided with an invert of paving brick. Segmental block sewers have been built ranging in size from 42 to 60-in. diameter, in several cases, where a dry and firm foundation rendered any concrete backing unnecessary. In all cases the blocks are required to be bedded in cement mortar to insure a firm bearing and to guard against the danger of leaky joints. No difficulty has been experienced with this type of construction and the resulting sewers are entirely satisfactory.

New Features in Concrete Spouting Plant

The new Field Museum of Natural History, in Grant Park, Chicago, will be an imposing marble structure carried on massive concrete piers with pile foundations. It will be 706 ft. in length, with a width of about 450 ft. for the central portion and 335 ft. for the end portions. The central portion will be about 75 ft. high from roof to ceiling, and the side portions will be divided into bays alternately one and three stories high, thus affording ample light for the comparatively narrow upper floors.

The structural design provides for a series of large monolithic concrete piers, which support steel framing for the reinforced-concrete floors. There will be no interior columns, the floors having long spans between the piers and the latter serving to divide up the interior space into separate but connected rooms. In order that the entire structure may be carried by these piers and thus be independent of any settlement of the surrounding ground, the exterior piers have cantilever brackets, which will support the floors and retaining walls of the areaways and entrances. Some of these piers and brackets are shown at the left in Fig. 4, which is a general view of the work (taken Nov. 5).

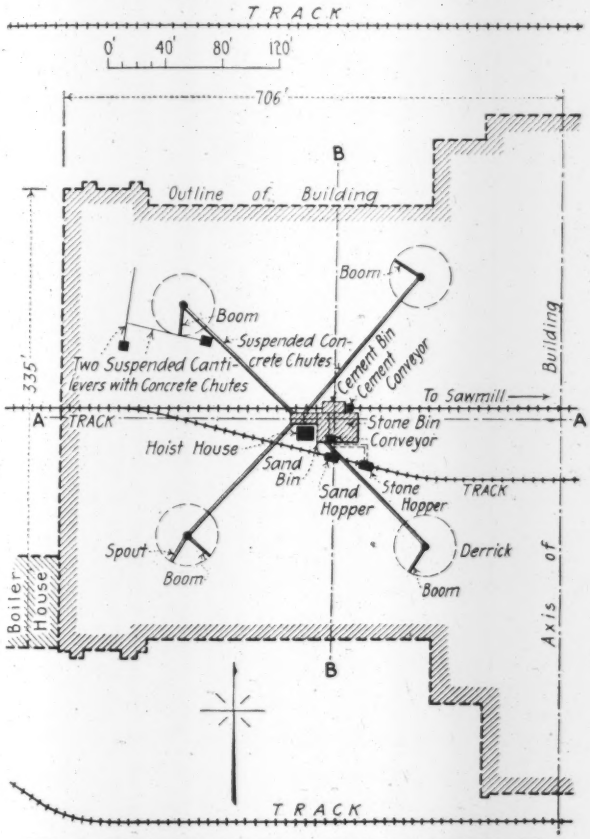


FIG. 1. PLAN OF CONSTRUCTION PLANT FOR THE FIELD MUSEUM, CHICAGO

The building is on made ground not yet filled to final elevation, and a special feature of the work is that the substructure, or basement, is being built above the present level of the ground. It will be covered later when this part of the park is filled and graded around the museum.

The site is land reclaimed by filling in along the lake front, a timber and crib bulkhead forming the outer or water line of the park. The depth of water was from 10 to 15 ft., and the present elevation of the site is about 9 ft. above water line. The elevations of the new building will be 34 ft. above water for the basement, 40 ft. for the terrace surrounding the museum and 50 ft. for the first floor.

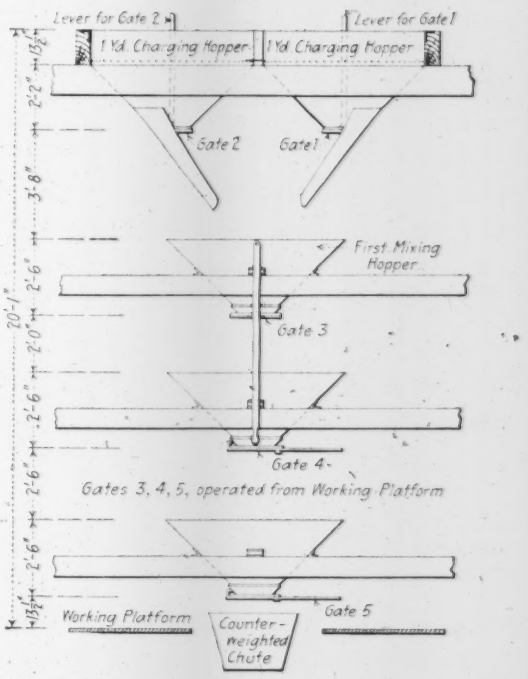


FIG. 2. SECTIONAL ELEVATION OF GRAVITY CONCRETE MIXER WITH DOUBLE CHARGING HOPPER

The entire area is underlain by the Chicago clay, which is mainly a stiff blue clay (with some softer streaks and pockets), changing to hardpan in the lower strata. The foundations are 60-ft. timber piles, grouped under the various footings and cut off at 21 in. below water level. They are driven by steam pile-hammers, long followers being used to put them down to the required depth.

The pier footings are staked out on the surface, and sheeting of single planks is driven to inclose the spaces. The ground is then excavated to and around the piles, the sheeting being braced as the excavation proceeds. The forms for the concrete are then placed. The base of the footings is 30 in. below the water level, or 9 in. below the pile heads.

Details of Concrete Mixing Plant

The construction of the substructure will be carried out in two sections, the west half being built first. The concreting plant is placed at the center of the west half, so that its spouting system can cover the entire area of this section of the work. This arrangement is shown by the plan, Fig. 3. Four railway tracks run across the site and are connected with the adjacent main line of the Illinois Central R.R.

Cement is delivered in bulk in box-cars and is plowed out of the cars by a man handling a scraper, which is operated by a cable and drum. The cement falls directly into the boot of an elevator. Sand and stone are deliv-

ered in drop-bottom cars and dumped at two track hoppers, each of which feeds a short lateral belt conveyor delivering the material to an elevating conveyor. The three materials are discharged into elevated storage bins having capacity for about 400 bbl. of cement, 200 cu.yd. of stone and 100 cu.yd. of sand. From these bins the materials are fed through gates to the measuring hoppers of the mixer.

The concrete is made in a Hains gravity mixer, consisting of three superimposed hoppers, as shown in Figs. 2 and 3. To expedite the work, this mixer is served by two charging hoppers, each of 1-yd. capacity and employed alternately. A man at the top operates the gates of the charging hoppers, and a man on the working platform below operates the gates of the three mixing hoppers in succession. From the last hopper the concrete flows through a chute to a 1-yd. elevator bucket. With the two charging hoppers the concrete can be mixed at the rate of 100 batches (or 100 cu.yd.) per hr.

The elevator tower is 8x16 ft., 175 ft. high, and the bucket is hoisted at a speed of about 300 ft. per min. by a Thomas electric hoist with a 100-hp. motor. The bucket dumps automatically (by tilting) into a hopper which can be shifted to any desired height in the tower. This hopper is of 2-yd. capacity, so that it can hold two batches if the distribution is temporarily checked. It has a four-way gate serving four lines of spouting. This gate is operated by a man at the hopper, his working platform being housed as a protection against the weather.

The system of distributing the concrete by gravity spouting is of special interest on account of its flexibility and wide range of reach, enabling practically all the concrete to be spouted directly into the various forms with very little use of wheelbarrows or hand carts. The entire distribution system, including the tower bucket and hopper, was supplied by the Insley Manufacturing Co., and is shown in Figs. 2, 3 and 5.

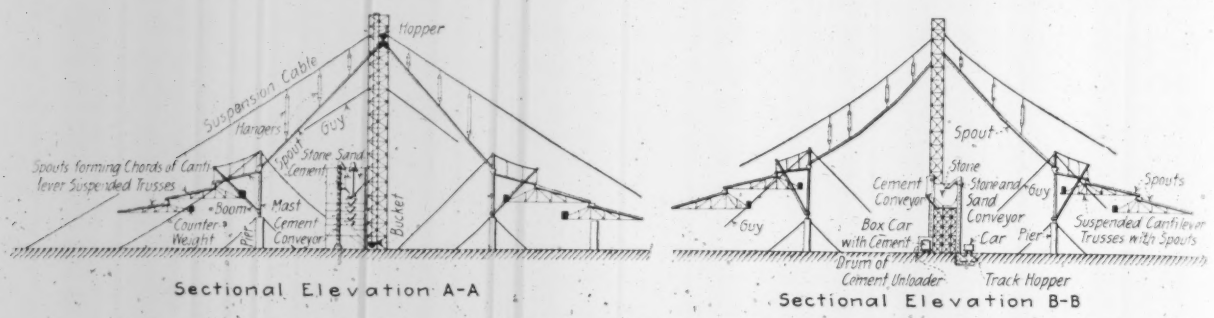


FIG. 3. CONCRETE-MIXING AND DISTRIBUTING PLANT FOR THE FIELD MUSEUM, CHICAGO

The tower is placed at the center of the site, Fig. 2, and from it extend four lines of steel chutes or spouts. These are supported by hanger tackles from anchored suspension cables, as shown. The hangers enable the height and slope of the spouts to be adjusted as required, the free

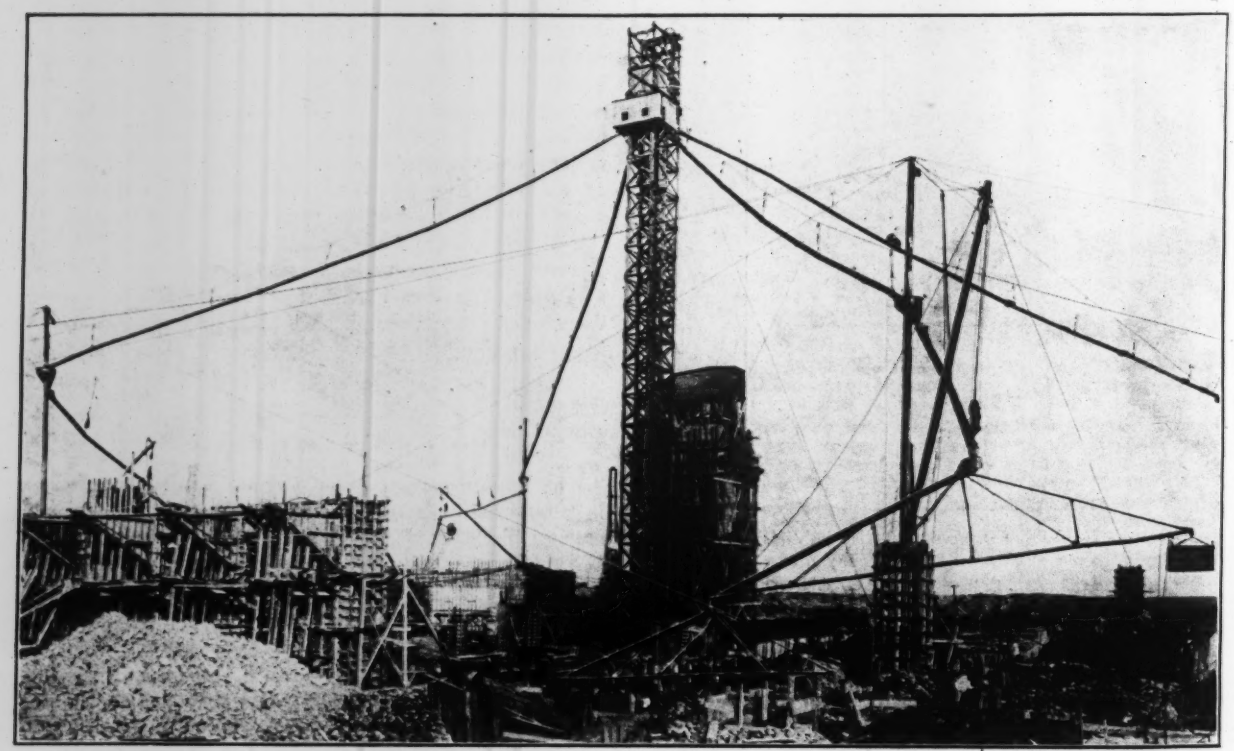


FIG. 4. CONCRETE-MIXING AND DISTRIBUTING PLANT FOR THE FIELD MUSEUM, CHICAGO

ends of the ropes being led to convenient points on the ground. The spouts are 150 ft. long, and each serves an articulated system of spouting, which can be swung in all directions.

At the end of each 150-ft. fixed spout is a guyed derrick mounted on a timber tower or (later) on one of the concrete piers of the building, about 30 ft. above the ground. Each derrick has a 60-ft. mast and 50-ft. boom and carries a spout 40 ft. long suspended by hanger tackles

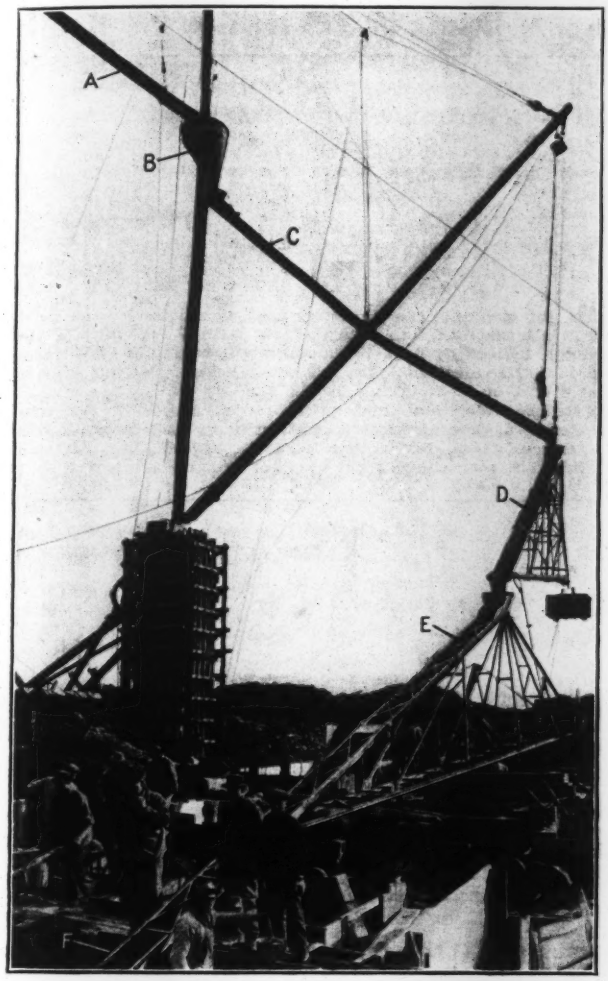


FIG. 5. FLEXIBLE SPOUTING SYSTEM FOR DISTRIBUTING CONCRETE

A—Spout from tower. B—Hopper sliding on derrick mast. C—Spout swung by derrick. D and E—Suspended cantilever trusses carrying spouts. F—Portable spout leading the concrete into the form

from the topping lift. The upper end of the spout has a conical hopper with side discharge; this slides on the mast (the mast passing through the hopper).

From the boom is hung a triangular cantilever truss 60 ft. long and 12 ft. deep. On the top chord of the longer arm is a 40-ft. length of spouting, while the end of the shorter arm carries a concrete counterweight. From the end of this truss is suspended a second and similar truss. This arrangement makes three 40-ft. lengths of spouting, with articulated connections so that each can be pivoted around the end of the one above.

With the swinging of the derrick boom and the trusses practically all points within a wide area can be covered. The derricks and main lines of spouting are shifted as required. Fig. 1 gives a general view of the mixing and distributing plant. Fig. 5 shows in more detail the flexible portion, which comprises the derrick chute and two swinging cantilever chutes.

The architects are Graham, Burnham & Co. The Thompson-Starrett Co. has the general contract, and Robert W. Grange, Jr., vice-president and Western manager, is in charge of the work.

Test Comparing Steam and Drop Hammer Pile Formulas

In the foundation work for the Pearl Harbor dry dock at Hawaii, Civil Engineer E. R. Gayler, U. S. N., had nine piles driven with a drop hammer and eleven piles with a steam hammer in an area having soil of uniform consistency and therefore presumably of uniform bearing value. From the data so obtained he made the following comparison of the accepted bearing-value formulas.

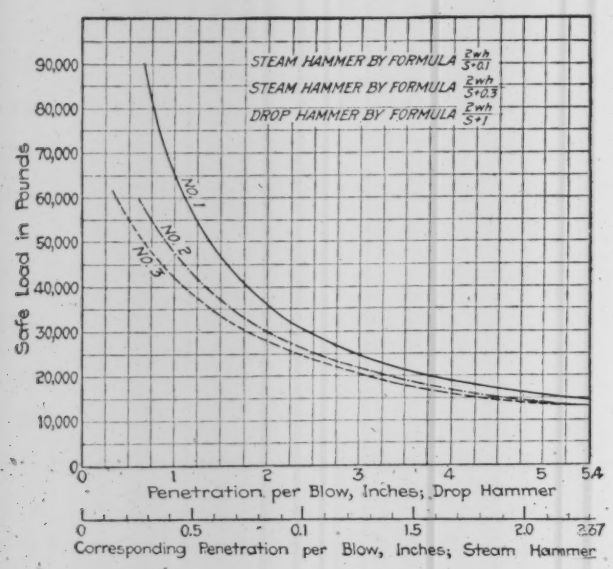
The drop hammer weighed 3,460 lb., and the fall was uniformly 12 ft. The steam hammer had a moving part weighing 5,000 lb. and a stroke of 3½ ft. The piles were of Douglas fir, 80 to 100 ft. long, with the small end sawed off square, and were driven in 40- to 50-ft. depths of water without the use of a follower. The tips of the drop-hammer piles averaged 8.1 in. in diameter, and the depth driven into the ground ranged from 18.2 to 48.9 ft., averaging 30.7 ft. The tips of the steam-hammer piles averaged 9.1 in. in diameter, and the depth in the ground ranged from 24.8 to 38.9 ft., averaging 30.1 ft.

The average penetration in inches per blow for each 4 ft. of driving is given in the accompanying table.

Depth in Ft.	Penetration in In.	
	Steam	Drop
0-4	2.37	5.4
4-8	1.46	3.7
8-12	1.29	3.2
12-16	0.86	2.38
16-20	0.77	1.79
20-24	0.55	1.14
24-28	0.37	0.85
28-30	0.29	0.27

The safe bearing values of the drop-hammer piles computed by the *Engineering News* formula $F = \frac{2wh}{s+1}$ and of the steam-hammer piles computed by the formula $F = \frac{2wh}{s+0.3}$ correspond fairly closely when reduced to corresponding penetrations, as shown on the accompanying diagram. By referring the drop-hammer values to what may be termed standard driving, using $w = 3{,}000$ lb., $h = 20$ ft., and s proportional to the observed penetration in the ratio of $\frac{3{,}000 \times 20}{3{,}460 \times 12}$, the accordance between the two sets of bearing values is considerably closer. When the steam-hammer pile values are computed by the formula $\frac{2wh}{s+0.1}$ (the generally accepted steam-hammer formula), the results vary considerably from the above and are on the unsafe side. Apparently the use of this formula leads to the bearing values that are excessive.

Somewhat similar conditions were found in driving test piles for the Mare Island dry dock in 1906, where the average pile penetration per blow under the 29-ft. fall of a 2,600-lb. drop hammer was 0.86 in., which corresponded to a final penetration of 0.3 in. for the steam hammer with a 5,000-lb. moving part and a 41-in. stroke. The safe load by the usual drop-hammer formula referred



COMPARATIVE BEARING VALUES OF TEST PILES WITH STEAM AND DROP HAMMERS

to a 20-ft. fall of a 3,000-lb. hammer is 70,000 lb. By the steam-hammer formula $\frac{2wh}{s + 0.3}$ the safe load is 58,000 lb.; by steam-hammer formula $\frac{2wh}{s + 0.1}$ the safe load is 87,500 lb., which is apparently excessive.

At Pearl Harbor the average penetrations per blow by steam hammer and by drop hammer at the same attained depth were almost exactly proportional to the respective energies of the blow. At Mare Island this relationship, as applied to the two kinds of hammers, was not so close, as the ratio of the respective penetrations was 0.35 and the striking energies 0.23. The steam hammer, probably because of its greater weight, gave about 50% more penetration in proportion to the energy expended; but the penetration, 0.86 in. per blow of the 29-ft. fall of the drop hammer, was very closely in proportion to the penetration 0.34 in. per blow of the same hammer with an 11-ft. fall.

NOTES

Area in Acres—Twice the area in square yards, and the decimal point shifted four places to the left (or, as the author states it, twice product of dimensions in hundreds of yards), gives the approximate number of acres. The result is 3.2% small, and may be corrected by adding 1/30 of the result. Thus, with an area a mile long and ¾ mi. wide, the dimensions are 1,760 yd. and 1,320 yd., or in hundreds of yards 17.60 and 13.20. The product is 232.32, and twice this is 464.64. Adding a correction of 1/30, or 15.488, gives 480.13 acres. The true area is 480 acres. This rule, given in "Professional Memoirs" of November-December, 1915, is credited to G. E. Gibson, Assistant Engineer, Barge Canal Office, Albany, N. Y.

Top Forms in Arch Concreting—Top forms are considered necessary by H. B. Watters, resident engineer on the new Chesapeake & Ohio Northern Ry. work at Sciotoville, Ohio. It has long been usual in concrete bridgework to omit top forms near the crown of the arch where the slope is very small, but of late it has become rather common practice to omit these forms far down the arch rib slope and to attempt to screed the top surface to line. Concrete so placed is apt to pile up, causing an excess of arch-ring depth at the bottom of the section poured and a thinning of the ring near the crown. To avoid this dangerous condition Mr. Watters uses a top form for the haunch section of the arch ring, which extends from the springing line to a crown section placed in advance about 3 ft. on either side of the crown.

Tables for Estimating Stone or Concrete Trim have been compiled by W. E. Corbett, of Corbett & Clarke, Hackensack, N. J., and are presented herewith. The figures in the first horizontal and the first vertical line in both tables are the cross-sectional dimensions of the construction to be estimated. The weight or volume per lineal foot of a piece of 6x6-in.

TABLE 1. WEIGHT OF TRIM, POUNDS PER LINEAL FOOT

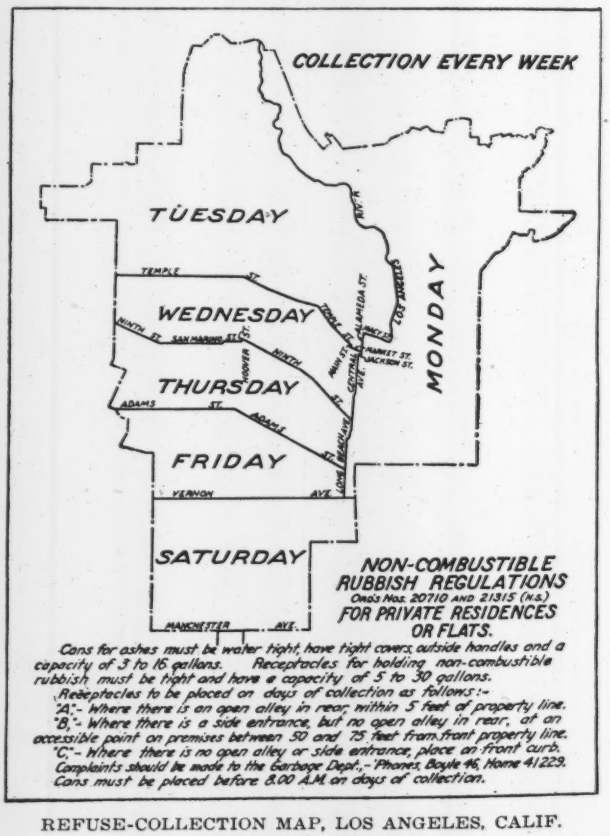
Sizes, In.	4 In.	5 In.	6 In.	7 In.	8 In.	9 In.	10 In.	11 In.	12 In.
4	17	21	26	29	33	38	41	45	50
5	21	26	32	36	42	47	53	57	63
6	26	32	38	44	50	57	63	69	75
7	29	36	44	51	59	66	74	81	87
8	33	42	50	59	66	74	84	92	99
9	38	47	57	66	74	84	93	104	113
10	41	53	63	74	84	93	104	114	125
11	45	57	69	81	92	104	114	126	138
12	50	63	75	87	99	113	125	138	150

TABLE 2. VOLUME OF TRIM, CUBIC FEET PER LINEAL FOOT

Sizes, In.	4 In.	5 In.	6 In.	7 In.	8 In.	9 In.	10 In.	11 In.	12 In.
4	0.11	0.14	0.17	0.19	0.22	0.25	0.27	0.30	0.33
5	0.14	0.17	0.21	0.24	0.28	0.31	0.35	0.38	0.42
6	0.17	0.21	0.25	0.29	0.33	0.38	0.42	0.46	0.50
7	0.19	0.24	0.29	0.34	0.39	0.44	0.49	0.54	0.58
8	0.22	0.28	0.33	0.39	0.44	0.49	0.56	0.61	0.66
9	0.25	0.31	0.38	0.44	0.49	0.56	0.62	0.69	0.75
10	0.27	0.35	0.42	0.49	0.56	0.62	0.69	0.76	0.83
11	0.30	0.38	0.46	0.54	0.61	0.69	0.76	0.84	0.92
12	0.33	0.42	0.50	0.58	0.66	0.75	0.83	0.92	1.00

trim, for example, can be determined by moving to the right on the horizontal line which starts with 6 in., until in the column headed by 6 in. the answer will be found—0.25 sq.ft., or 38 lb., as desired. The weight table is prepared on the basis of 1 cu.ft. weighing 150 lb.

Refuse-Collection Card with District Map—For the convenience of householders a district map is printed on the refuse-collection instruction cards issued in Los Angeles, Calif. The map shows what parts of the city are served each


COLLECTION EVERY WEEK
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY
MONDAY
TEMPLE ST.
NINTH ST.
ADAMS ST.
VERNON AVE.
MANCHESTER AVE.
NON-COMBUSTIBLE RUBBISH REGULATIONS
Ords Nos. 20710 AND 21315 (N.S.)
FOR PRIVATE RESIDENCES OR FLATS.
Cans for ashes must be water tight, have tight covers, outside handles and a capacity of 3 to 15 gallons. Receptacles for holding non-combustible rubbish must be tight and have a capacity of 5 to 30 gallons.
Receptacles to be placed on days of collection as follows:-
'A'- Where there is an open alley in rear, within 5 feet of property line.
'B'- Where there is a side entrance, but no open alley in rear, at an accessible point on premises between 50 and 75 feet from front property line.
'C'- Where there is no open alley or side entrance, place on front curb.
Complaints should be made to the Garbage Dept.,- Phones, Boyle 46, Home 41223.
Cans must be placed before 8.00 A.M. on days of collection.

REFUSE-COLLECTION MAP, LOS ANGELES, CALIF.
(The original was 5x6½ in.)

working day of the week under the prevailing weekly collection of noncombustible refuse. The card also gives excerpts from the ordinance governing receptacles for ashes and for rubbish.

Editorials

A Railway Freight Blockade Without Precedent

Nine years ago, in the winter of 1906-7, a congestion of railway freight traffic occurred which extended over a large part of the country and produced the most serious freight blockade ever recorded in American railway history. The chief cause of that blockade was the fact that in many parts of the country railway terminal facilities were inadequate to handle the extraordinary volume of traffic which was thrust upon them during a period of unprecedented business prosperity. Secondary causes were that many railways were short of motive power at that time and short also of freight rolling stock.

On many of the railway lines east of Chicago the blockade of 1906-7 is being repeated today, as a result, however, of causes entirely different from those which produced the blockade of nine years ago. There has been for the past twelve months a very heavy railway traffic from interior points to Atlantic Coast ports for export shipment. In order that the cars carrying this export freight may be promptly unloaded when they reach the port for which they are destined, ships must be ready there to receive it.

Ever since the outbreak of the war the demand for shipping facilities has been greater than the available ocean vessels could supply. The National Foreign Trade Council recently estimated that the total tonnage of merchant vessels rendered unavailable for private commerce since the war began is between 20 and 25% of the total tonnage at the outbreak of the war. To some extent this reduction in the available shipping has been offset by a reduction in the amount of international trade. It must be remembered that great as is the amount of traffic now passing from America to Europe, there are other ocean shipping routes on which trade is at present far below the normal.

It is obvious, however, that there are not enough ships to supply the demand, and the condition is aggravated by the delays in unloading vessels in ports on the other side of the ocean.

In no industry are prices more promptly responsive to the law of supply and demand than in the ocean-freight business, and rates charged for ocean carriage are double and triple, in some cases even five to seven times what they were before the outbreak of the war. Even at these rates vessels cannot be obtained fast enough to supply the demand at the busiest Atlantic seaports, and for some time cars have had to wait weeks and even months before room could be found on board ship for their contents.

Fortunately the railway managers learned a lesson from the experience of nine years ago, and they put in operation, none too soon, an embargo against further shipments of freight to the points of congestion. Experience has abundantly demonstrated that if a railway allows its yards to become filled with cars beyond the point where free movement can take place, traffic will soon come to a standstill. The railway must keep tracks enough open for the movement of its passenger traffic and for the handling of perishable freight and livestock, and for the delivery to the communities along its line of such essentials of daily living as milk, meat, flour, fuel, etc. If it allows other classes of freight to accumulate in its yards, it will soon create just such a deadlock as afflicted a number of prominent railway centers in 1907.

It is an interesting question whether the railway facilities of the present day would be sufficient for the present volume of traffic if sufficient ships were available at the seaports promptly to take care of the export business. There has been, it is true, an enormous extension of terminal facilities since 1907. Had there not been, the natural growth of traffic since that time could not possibly have been taken care of. During the last two or three years, however, the prevailing sentiment in railway circles has been that fully as much money had been invested in terminals as business would warrant, and the long periods since 1907 when there has been an enormous surplus of unused cars have had a tendency to keep down investments in rolling stock.

It is of interest in this connection to note that in an official statement addressed to the Interstate Commerce Commission on Dec. 28, the New York, New Haven & Hartford R.R. declares that it is doing the largest volume of business in its history and has reached a point where its present physical facilities make it unable to handle a larger volume of traffic. It frankly admits its responsibility to the public for increasing these facilities, and announces that improvements to its lines costing $5,000,000 will be made as rapidly as men, material and weather will permit; that it has 33 heavy freight locomotives to be delivered in January and February and 500 coal cars in March, and that it is ready to spend in addition very large sums for more tracks, bridges, terminals, shipping facilities and motive power if money can be obtained.

Railway Boat Lines

As many of our readers will recall, a clause in the Panama Canal act forbids a railway company to operate vessel lines directly or indirectly, except by special permission of the Interstate Commerce Commission. The reason for this provision was the idea that if the railways were allowed to run their own steamship lines through the Panama Canal they would first cut rates to a point which would ruin competitors and then raise them to such a figure that the canal route would not seriously compete with the rail lines across the continent. It is claimed that such tactics have been actually used by coastwise steamer lines operated in connection with railways to drive off competition.

Whatever of truth there may have been in this argument in the past, experience of the last few years appears to demonstrate that independent transportation companies which engage in coastwise traffic do not fur-

nish facilities and service equal to that given by the railway-owned steamer lines. Shippers and travelers prefer to patronize a boat line run by a responsible railway company in preference to a line of whose responsibility they know little or nothing.

For many years the service on Long Island Sound furnished by the boat line operated by the New Haven R.R. was famous for its excellence. In recent years, as might naturally be expected, the railway company has given little attention to its boat service. It has not, however, ignored the possibilities of development in this direction, for President Elliott says, in the statement above referred to:

> The company has tried to relieve its rail lines so far as it could by diverting business to its boat lines, but here again it has not been able to add to its facilities because of the uncertainty surrounding the future of the boat lines. The company considered very seriously placing an order nearly a year ago for two additional freight steamers to run on Long Island Sound, but it did not feel that it had the right to borrow the large sum of money necessary, considering the fact that within a comparatively short time the company might be ordered, under the Panama Canal Act, to dispose of all of its water lines. The relief, therefore, given by the boats is not as great as the company would like.

There is a reason why capitalists naturally hesitate before investing a large amount of money in coastwise steamship lines to compete with railways. In order that any shipping line shall do a profitable business it must have the coöperation of connecting railway lines. It is probably within the power of state and interstate commissions to compel railways to grant certain facilities, such as through routes and through rates, to a coastwise steamship line; but coöperation produced by compulsion is not apt to be effective and valuable coöpertion. The law can compel certain acts to be performed, but it cannot possibly deal with the thousand and one details which are necessary if a railway is effectively to aid a steamer line to get business instead of encouraging travelers and shippers to use the rail line.

On the other hand, if a railway company were permitted to operate steamship lines, there is certainly sufficient authority in government supervising commissions to see that proper facilities are furnished and at reasonable rates.

An Engineer Reviews His Work

Few men who give their names to a marked engineering advance are able to lead that advance so far and so ably as has Dr. Karl Imhoff. Eight years ago, realizing some of the drawbacks to the septic-tank process of sewage clarification and sludge reduction, and with keen appreciation of the improvements in this process suggested by the Massachusetts State Board of Health and partly worked out by Dr. Owen Travis, Dr. Imhoff designed a new type of two-story tank. As engineer of the Emscher Sewerage District, in Germany, Dr. Imhoff had immediate opportunity to install a number of these tanks. Promptly and enthusiastically introduced into the United States by Dr. Rudolph Hering, the Imhoff, or Emscher, tank has since been widely used here, although not so extensively as in Germany. Today a quarter-thousand Imhoff tanks are in use or under construction in the two countries.

Just how many of these tanks have been designed by or under the more or less close supervision of Dr. Imhoff cannot be stated. Unquestionably he has had some part in a large percentage of them, and engineers who have been inspired by personal touch with him have designed many of the others. Broadly speaking, the type has been followed more closely than is usual in the absence of rigid proprietary control. The large number of two-story tanks which he has himself designed, his closeness of touch with the work of others and the widespread and growing use of the Imhoff tank combine to give great interest and value to his review of "Eight Years of Imhoff Tank Design and Operation," begun elsewhere in this issue. The review is worth reading by not only those interested in sewage disposal, but also by engineers of sufficient breadth to desire to follow the outworking of a problem in engineering design and operation.

Engineers and Bridge Art

For an engineer to pretend to have any authoritative knowledge of art in construction is of course an impertinence. That privilege is the prized possession of the allied profession of architecture. The appearance of a bridge, however, is so important for its success that an engineering journal may presume to offer some humble comments upon it.

On the first page of this issue is a view of the Q St. bridge in Washington, D. C., the latest of several arch bridges which span Rock Creek in that city. In such a black and white print the colorful tones of the bridge are lost and the prodigality of decorative detail can hardly be appreciated. This is unfortunate, for it is in these two features that the bridge differs most radically from other bridges which have been built by engineers without the aid of architects. In the Q St. bridge the decoration of the bridge was intrusted to an eminent architect.

There is a widespread impression that engineers are purely utilitarian, that they are too much concerned with strength and economy to consider appearance and that, in the minds of the profession, strength and ugliness are twin sisters. There may have been some ground for this impression years ago, particularly in steel-bridge design, but in the design of the masonry arch most engineers have learned that simplicity generally means beauty and that that arch which best satisfies the structural requirements will as a rule best satisfy the eye. Most engineers therefore have hesitated to attempt to improve on the simple lines and tones of the concrete arch by either decoration or coloring.

In the Q St. bridge, however, an architect has attempted what no mere engineer would have dared. There are an elaboration and a profusion of ornament and tone which may possibly delight the artist or the architect, but which are not altogether pleasing to the engineering eye, accustomed to simplicity of detail. There is of course ample precedent in Europe for profuse bridge decoration. Practically every one of the many bridges crossing the Seine in Paris is bedecked with statues and basreliefs. Artists and art critics frequently bewail the absence of similar decorative bridges in American cities, which might lead one to believe that the American engineer is too commonplace in his ideas and that a conception of the beautiful is as far from his ability as the production of it.

It will hardly be disputed, however, even by the artists and art critics, that if the choice in bridge architecture

between simple plain outlines and a profusion of over-decoration, the simple style is far preferable. An engineer viewing the Q St. bridge will generally have no regrets that he has not ventured to hang decorations on his bridges. To his possibly untrained eye, the coloring is too emphatic, the decorations too obtrusive and the arched sidewalk brackets too fantastic. When compared with the simple lines and tones of the nearby Connecticut Ave. bridge—also artistically supervised by an architect—the newer bridge appears almost tawdry.

Individually each buffalo on the Q St. bridge seems to be a pretty good animal as bronze animals go, but collectively there are too many of them and they are too large for comfort. The impression is unavoidable that a whole herd of this nearly extinct species has selected an entirely unsuitable feeding ground and that a very small bridge has been inadvertently left underneath some very large bison. Incidentally, those who have wondered why animals—and wild animals at that—are artistic necessities of bridge decoration may be interested in the bridge pictured on page 40 of this issue. The Chinese school of art—wild dogs, sculptured lions and all—has evidently bridged both time and space and is influencing our modern architects.

Developments in Surveying

It is not very frequently that the editor can direct attention to new methods of surveying, but on another page the reader will find an article describing some truly radical improvements in the field work of topographic surveying. While this article deals specifically with reconnoissance surveys for logging operations, it will be readily seen that the method is much more accurate than work hitherto done for this purpose, and there is no reason why the same methods cannot be applied to railway reconnoissance or to any survey requiring topographic details of ordinary accuracy.

The new method involves the use of a special Abney clinometer and a specially graduated tape. With this clinometer the vertical rise per chain (66 ft.) is read directly, and used in conjunction with the special tape, it permits stakes or markers to be set at unit intervals of horizontal measurement, quickly and without computations or the use of tables, irrespective of the slope to which the tape is applied.

Here are all the elements necessary to locate contours except the horizontal control. In thickly wooded forest land this control is obtained by a series of parallel lines run out by a compass, but when desirable, any other system of control may be adopted. Experience shows that elevations may be made to check within 1 ft. per mi. (10 ft. per mi. is the average for rough work), so that a more accurate system of control than compass lines is justified in an open country.

Incidentally the attention of the reader is called to the articles in *Engineering News,* June 10 and Dec. 16, 1915, which give in considerable detail the methods used and the accuracy obtainable by the use of an aneroid barometer for topographic reconnoissance. The former article describes the use of aneroids for forest surveys and the second article describes the use of a single aneroid for a very rough reconnoissance. These two articles and the one in this issue comprise a very complete and uptodate treatment of this important branch of surveying.

Model-City Planning Boards

If cities are to be planned instead of happening, someone must be charged with the responsibility of the task. A half-dozen years ago Hartford, Conn., under the leadership of its city engineer, created the first city-planning commission in the United States. Today there are perhaps a hundred such commissions, East and West, North and South. Naturally in the earlier unchartered stages of city planning control and under our multifarious forms of city government, the composition, powers and duties of American city-planning commissions and the relations of such commissions to other branches of the city government were most varied. It will take some time to find out just what is the best method of control—or perhaps the three or four best methods to suit varying conditions.

An attempt to outline a scheme for city-planning boards under the commission-manager form of city charter has just been made by a committee of the National Municipal League. Without repeating the details, printed elsewhere in this issue, attention may be called to these important features of the scheme of particular interest to engineers: (1) Everything affecting the city plan must be passed upon by the board. (2) Every year the board must submit a city-planning program for at least three years to come. (3) The city engineer is recognized as the chief city planner in ordinary, so to speak, but the board must employ as secretary a person skilled in city planning and may engage the services of consulting experts in city planning. All these features are commendable.

The engineering profession throughout the country will heartily approve the selection by Governor Whitman of Henry W. Hodge as a member of the New York City Public Service Commission. For several years engineers have discussed the need of appointing men of engineering experience and demonstrated executive ability as members of state public-service commissions, and organized efforts to secure such appointments have been made by the engineering societies. The selection of Mr. Hodge for this position by Governor Whitman is in line with these recommendations of the profession. More than all else, the public is to be congratulated that an office of such importance is to be filled by a man in every way competent to discharge its duties.

The letter from General Kingman, on the next page, reports a definite step forward in the Corps of Engineers of the United States Army, a step which must be pleasing to the civilian engineers of the country and one that has been urged in public and in private for many years. It is true that some years ago the way was made clear to permit the commissioning of capable young engineers from civilian life, but the rarity of such appointments was such as to foster the suspicion that the more-or-less intentional choking off of civilian recruiting was to extend from the civil-engineering corps of the navy to the similar corps in the army. The success of the latest army examinations and the methods used to get candidates for those examinations show that the present administration of the Engineer Corps at least is determined to get good men where they can be found and not to depend alone on West Point, which, at best, is not an engineering school.

Letters to the Editor

Sharp Thread Caused Failure of Suspender Rods

Sir—I have been interested in the description of the failure of the Division St. Bridge at Spokane, Wash., as published in your issue of Dec. 30, 1915, and think that Hugh L. Cooper has probably reached the correct conclusion—that the cause of failure was the treatment of the eye-bars after the accident of 1894.

I would call attention, however, to the comparison in your editorial to the breakage of suspender rods in the Brooklyn Bridge. The breakage of these rods, as I remember, was not due to any heat-treatment after they were manufactured, but rather to the improper shape of the screw thread as originally cut. These suspenders were among the first to be made of steel in bridge construction and before it was generally recognized that a sharp cut in homogeneous steel is equivalent to an incipient crack that will creep through the steel when the metal is repeatedly strained. The threads were cut with sharp angles (no fillet at bottom), and the fracture showed by concentric rings of rust that the suspenders had broken gradually until so small a section remained intact that it could not then stand the load imposed.

When one suspender broke, it increased the load on adjacent suspenders, and several others broke in succession.

HENRY B. SEAMAN.

Woolworth Building, New York, Dec. 31, 1915.

Six Civilians Commissioned in Army Engineer Corps

Sir—I take pleasure in sending you the following information:

As a result of the examination for appointment as probational second lieutenant in the Corps of Engineers, United States Army, held on Oct. 25-30, 1915, the nominations of the six successful candidates are about to be sent to the Senate. These are Messrs. Gilbert D. Fish, New York City; George Mayo, Washington, D. C.; Paul T. Bock, New York City; Charles O. Boynton, San Francisco, Calif.; Arthur P. von Deesten, Brooklyn, N. Y.; and Edward N. Whitney, Madison, Wis. Mr. Boynton is a junior engineer who has been several years in the Government service, under the Engineer Department.

The number of candidates who actually took and completed the examination was 17. In addition three candidates withdrew during the progress of the examination, while two were found physically ineligible as a result of the physical examination which preceded the mental examination.

The nomination of the six successful candidates marks the completion of a vigorous effort on the part of the Chief of Engineers of the army to secure thoroughly well-qualified young engineers for appointment to the Corps of Engineers under the provisions of the river and harbor act approved Feb. 27, 1911.

The number of candidates presenting themselves for this examination in previous years was disappointingly small, and very few of the candidates who presented themselves had the necessary qualifications. In the period from January, 1912, to February, 1915, four examinations were held. The total number of persons who took the examinations was 11; of these three were commissioned as probational lieutenants.

In order to secure a larger number of candidates, the Chief of Engineers on Apr. 1, 1915, addressed letters to the presidents of all the approved technical schools, about 50 in number, requesting that they draw the attention of suitable graduates of their institutions to the opportunity for appointment as lieutenants in the Corps of Engineers. The Chief of Engineers also issued instructions to all of the District Engineer officers in the United States to facilitate in every way the candidacy of junior engineers in the employ of the Engineer Department and of any other candidates who might make inquiry regarding the examinations.

Arrangements were made with the Civil Service Commission by which a special examination was held to permit candidates to establish eligibility for junior engineer as required by law.

As a result 25 persons were authorized to take the examination beginning Oct. 25; of these, two withdrew before the examination and one failed to appear, leaving 22 who presented themselves for the examination, as already indicated.

It is anticipated that the next examination for appointment of the Corps of Engineers will be held at a convenient time after the termination of this college year.

DAN C. KINGMAN,
Chief of Engineers, U. S. Army.

Washington, D. C., Dec. 31, 1915.

Dishonest Hydraulic-Turbine Specifications and Bids

Sir—Bidders on turbines for hydraulic-power development are often asked to submit with their bids the "estimated approximate weights of the turbine units." Where, as is commonly the case in present practice, the equipment is specially designed, it is frequently impossible for the turbine manufacturer to estimate with precision the weights of the turbines at the time the bids are made. There is a temptation for the turbine manufacturer to overestimate the weights, for the reason that the purchaser is prone to think that the greater the weight of the machinery, the more he is getting for his money.

Every intelligent engineer will admit that this is a fallacy with reference to the purchase of almost any kind of high-grade machinery. It is certainly a fallacy in the case of hydraulic turbines. A certain amount of weight is absolutely necessary; but beyond this, if the design is correct, additional weight in the turbine runner, casing or gate mechanism is decidedly a disadvantage. Unneces-

ary metal in the runner increases the load on the bearings and is quite certain to reduce the efficiency. Unnecessary weight in the gate mechanism increases the cost of governors and also increases the difficulty of close speed regulation. Unnecessary weight in the turbine casing or in any of the other parts increases the size of crane necessary to handle the weight; increases the cost of freight and installation, and in many cases increases the power-station building cost throughout.

The excessive weights can be dishonestly employed to throw a turbine contract to favored builders. Now it must be admitted that engineers from time immemorial have been accustomed to resort to devious, and sometimes highly questionable, methods to eliminate undesirable bidders on contract work, though such practices have been carried out from good motives; identically similar methods may be utilized unfairly to throw work to favored bidders. In the case of hydraulic turbines, acceptance of the bid is often made contingent on several factors: First, efficiency, which may be estimated in various ways; second, net price bid. This as ordinarily interpreted would be supposed to mean the gross sum bid for the work. But suppose that one bidder overestimates the weight of the equipment, say 40%, not really intending to put this additional weight into the work, and that the other bidders estimate the approximate weights within say 10% of the true weights which they will furnish. The result is likely to be that the bid which is based on excessive weights will show an apparent cost per pound lower than that which any bidder whose weights are honestly given can afford to bid. The work may then be awarded to the highest bidder on the ground that the net price per pound is lowest, even though the efficiency guarantee is not above that of other bidders whose gross price is lower.

Examples of just this practice have come to the writer's attention, and it has seemed worth while to suggest a remedy. While a prospective bidder may not be able to tell in advance the exact weight within say 10% or 15%, he can at least tell what the minimum weight necessary will be. Accordingly it would seem that hydraulic-turbine specifications should require that bidders guarantee to furnish equipment having a certain minimum weight; and in order to prevent the furnishing of equipment of excessive weight, they should also be called upon to guarantee that the weight will not exceed the guaranteed minimum by more than a certain percentage, 10%, 15% or 20%, according to conditions.

This plan will at least furnish a uniform scale of comparison between bids, while under the present practice several bidders equally honest in their intentions may add various percentages to their estimated weights in order to be on the safe side. If this practice was generally followed, then the omission of such a weight guarantee clause from the specifications would in effect serve notice upon those proposing to bid on the work that there is likely to be an Ethiopian in the hedgerow.

CONSULTING ENGINEER.

A Coal-Thawing Shed will be built in connection with the coal-shipping pier of the Baltimore & Ohio R.R. at Curtis Bay, Baltimore, Md. Here cars of coal will be placed, in cold weather, before being run out to the car-dumping machines on the pier. The pier will have a capacity for shipping 10,000,000 tons per year and will cost about $1,500,000. It is to be completed before the end of this year.

Centrifugal Sewage Pump Saves Force Main

BY GEORGE G. EARL*

The new electrically driven centrifugal pump at New Orelans Sewerage Station A (designed by A. B. Wood, Mechanical Engineer of the Sewerage and Water Board, and purchased under the same contract as the large screw pumps for drainage) has recently been found to develop a capacity of 55 to 133 cu.ft. per sec. against heads of 43 to 72 ft. at efficiencies of 69 to 84%. This pumping unit has postponed for some years the construction of an additional force main. It is driven by a two-speed induction motor, takes water from the main sewer and discharges it through a 48-in. force main, 7,000 ft. long, into the Mississippi River. At low speed it makes 248 r.p.m. and discharges, depending upon the elevation of water in the river and in the sewer, from 55 to 85 cu.ft. per sec., showing an efficiency ranging from 69 to 83% on lifts (including friction in the discharge main) ranging from 43 to 35 ft. At high speed it makes 364 r.p.m. and discharges, depending upon the elevation of water in the river and in the sewer, from 106 to 133 cu.ft. per sec., showing an efficiency ranging from 80 to 84% on lifts (including friction in the discharge main) ranging from 87 to 72 ft.

These efficiencies are based in this case on heads shown by the pressure difference on the two sides of the pump and are slightly higher than was expected with a pump designed for two different speeds and two different conditions of lift, widely separated.

How Equipment Is Saved

There were originally installed at this station two engine-driven centrifugal pumps which were each capable of discharging 58 cu.ft. per sec. through the 48-in. force main with 51% efficiency. It was intended in the original design of the station to add later another steam-driven pump of larger capacity and to lay an additional force main to the river, since the original engines could not work together with the existing force main, thus bringing the capacity of the station to something over 128 cu.ft. per sec. with one engine and pump in reserve. This would have cost, for force main and additional engine and pump of 70 cu.ft. per sec. capacity, at least $135,000.

By substituting the new pump and motor at a cost of $29,000 we have added a unit whose capacity can be either 75 or 120 cu.ft. per sec. and which is of much higher efficiency than the existing steam pumps, and postponed for several years the construction of an additional force main which would have cost about $100,000 for a 48-in. main. The crowding of 106 to 133 cu.ft. per sec. through the existing force main creates a materially higher lift than would have been practicable for an engine-driven single-stage centrifugal pump, but the efficiency of the electrically driven unit operated from a central power station is so high that this additional lift requires little if any more fuel than would have been required if the additional outlet line and a steam-driven pump had been adopted. The need for the high-speed drive exists mainly during and immediately after storms, when the sewers are overcharged with ground water. The low speed is sufficient for dry-weather flow.

*General Superintendent, Sewage and Water Board, New Orleans, La.

Bridge Art at Its Origin

The arch bridge pictured on this page is one of many years' standing, located in the Imperial Summer Palace near Peking, China. It is shown here as an example of old Chinese bridge architecture, and incidentally to indicate a possible source of the wild-animal style of bridge decoration so well exemplified in the first article in this issue. The Chinese wild dogs compare favorably with the Washingtonian buffalo.

Public Concern in Water Power*

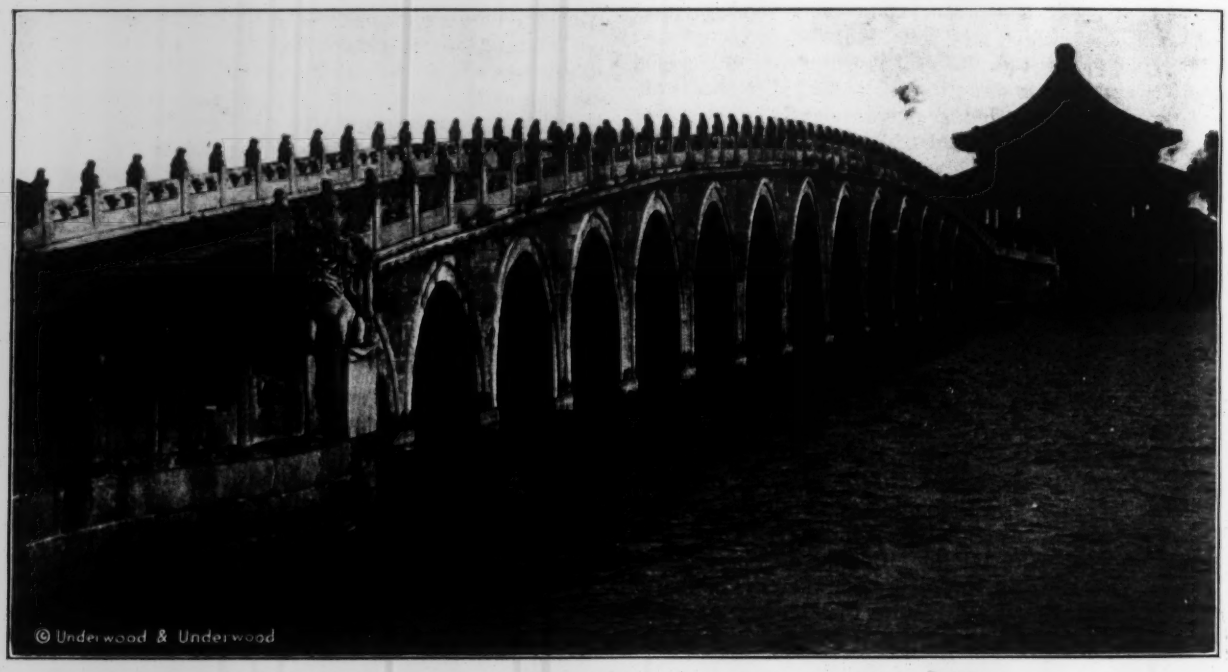
By George Otis Smith†

The people's interest in water power is served only through use. Whenever the energy of flowing water can be put to work at a cost justified by results, nonuse of a water power is an economic waste. But better not even a partial development now, if thereby the full utilization that may be needed in the near future is blocked. From the standpoint of the public what is needed in water-power utilization is efficient development that will meet market demands, and effective regulation that will secure to all parties interested a participation in the benefits of that development.

It is this logical desire for the best use, in which the largest possible number may share, that makes the utilization of water power a live issue. Unwarranted fear of impending monopoly may have led some to urge a halt in all development, while blind disregard of a changing order of things has caused others to denounce any restriction.

The engineering fact that stands out in the utilization of water power is the comparatively high efficiency of large systems. The combination of several water powers differing in character in order to serve many markets having diversified service requirements makes possible an economic adjustment of supply to demand. Nature's irregularities are thus equalized and man's varied requirements are similarly balanced. The Western states, in which these large hydro-electric systems have been mainly developed, show the largest per capita consumption of power and the lowest average rates.

In Montana one mining company by substituting hydro-electric power for steam effected an annual saving of nearly two million dollars. This means a saving of about 50c. on a ton of ore, thus permitting the mining with an equal profit of ore that much lower in grade than could be mined when steam-generated power was used. This instance illustrates the wide public interest that may be served through the efficient operation of large units. There exists a certain community of interest between the power corporation and the mining company. A similar community of interest needs only to be created between the public-utility company and the people it serves.



MARBLE BRIDGE IN IMPERIAL SUMMER PALACE AT PEKING, CHINA

The oft-debated question whether the relation of the Federal Government to water-power sites is that of proprietor or that of sovereign is only incidental to the real issue of the people's larger interest as the consumers of power. Herein lies the purpose of certain legislative proposals that have been misunderstood by some and perhaps misrepresented by others. The method of control may be based upon public ownership of land, police powers or preservation of navigation; the primary results sought will be public protection.

Too often in the past the wealth won by the engineer from the mine, the forest or the stream has been monopolized by the capitalist. Experience is teaching the American people to regard industrial development as a public

*From an address at the second Pan-American Congress, Dec. 27, 1915, to Jan. 8, 1916.

†Director, United States Geological Survey, Washington, D. C.

matter in which others than owners of stocks and bonds are concerned.

Too often the public interest in water-power resources has been discussed almost wholly in terms of compensation, tolls and charges for the publicly owned resources utilized. The revenue idea connected with this conservation issue has been allowed to overshadow the thought of control in the public interest. A notable exception to this tendency is contained in the Pend d'Oreille permit issued in 1913 by Secretary Lane, which provides that after the ten years allowed for construction of plant and development of market the rate of compensation to the Government shall vary inversely as the square of the proportional development of the site and directly as the square of the average price charged for electric energy to both customers and consumers. Here the objects plainly sought are prompt and complete development and low rates to the public rather than revenue to the Government.

When applied to water-power resources, as to other public utilities, regulation by public agencies needs to begin in the initial stages of development. The terms of the contract between the public consumer of electric energy and the private builder of the hydro-electric plant can best be considered before a stone is laid.

In any discussion of the future conduct of the water-power business one of the essential premises is the condition of proper return. This idea is at the foundation of public regulation of private business; but even beneath this is the query, A fair return upon what? The present trend in the determination of the proper basis for rate regulation appears to be away from valuations derived either from cost reproduction or from capitalization and toward actual legitimate investment.

The practical ideal in public regulation is to allow such returns to the private owner and operator as may encourage engineering and executive efficiency. To fix a maximum rate of profit based simply on capital investment might effectually veto improvement in plant or operation beyond the dead-line thus established; therefore the prize of a larger return must be held out to those who make the largest and best use of the natural resource. In the absence of the competitive element the returns to be derived by the private holder of a natural monopoly should be restricted so far as they depend primarily upon his use of the resource which society owns or to which society gives value; but a quite different method of regulation may well be applied to the profits that come from his continued investment of gray matter. Such ability and industry need not be capitalized, yet betterment of service should produce benefits in which both investor and consumer may share.

Adequate incentive to greater efficiency combined with full recognition of the people's interest may be provided by any of several methods—the sliding-scale plan adopted for the Boston gas companies makes larger dividends to stockholders contingent on lower prices to consumers; the Chicago street-railway ordinance provides for the division of profits above the fixed rate between the company and the public; a profit-sharing plan worked out by the manager of Wisconsin power company divides excess net profits equally between the company and the consumers. In each of these plans the principle is the same—the investing public and the consuming public are partners in the enterprise, and the duty of the Government is to exercise its protective power in the interest of the private owner of a public utility no less and no more than in the interest of the public user of that utility.

City-Manager Plan Adopted by 75 Municipalities

Large and small, 73 cities and towns in the United States and 2 in Canada have adopted the city-manager idea of municipal government. By this time the number may be larger, since the list published herewith was compiled some weeks ago and accessions are made at frequent intervals.

Two facts not disclosed by the table are that most of the places in the list combine the city manager and commission plans, and many of the cities and towns employ engineers as city managers.

MUNICIPALITIES UNDER CITY-MANAGER GOVERNMENT IN THE UNITED STATES AND CANADA

(As Reported to Ossian E. Carr, Secretary City Manager's Association, Up to November, 1915)

City	Population 1910	Date in Effect		Manager	Annual Salary
Norwood, Mass.	8,014		1914	C. A. Bingham	$3,000
Niagara Falls, N. Y.	30,445	Jan.	1916	O. E. Carr*	5,000
Newburgh, N. Y.	27,805	Jan.	1916	F. C. Alber*	6,000
Watertown, N. Y.	26,730	Jan.	1918		
Grove City, Penn.	3,674		1914	J. S. Ekey	2,000
Titusville, Penn.	8,533	Dec.	1913	H. A. Holstein	
Charlottesville, Va.	6,765		1913	A. V. Conway	
Fredericksburg, Va.	5,874			R. S. Royer	
Luray, Va.	1,218			Walter Campbell	
Portsmouth, Va.	33,190	Sept.	1916		
Staunton, Va.	10,604		1908	S. D. Holsinger	2,500
Wheeling, W. Va.	41,641	Jan.	1917		
Durham, N. C.	18,241				
Elizabeth City, N. C.	8,413	Apr.	1915		
Hickory, N. C.	3,716	May	1913	S. C. Cornwell	2,000
Morgantown, N. C.	2,712	May	1913	R. W. Pipkin	1,200
Thomasville, N. C.	3,877		1915	F. D. Jones	
Beaufort, S. C.	2,486		1915	H. C. Otis	
Rock Hill, S. C.	7,216		1915	J. G. Barnwell	2,400
Sumter, S. C.	8,109	Jan.	1912	Vacant	3,300
Lakeland, Fla.	3,719		1914		2,100
Largo, Fla.	291		1913	J. T. Sauls	600
St. Augustine, Fla.	4,104		1915	W. L. Miller	3,600
Johnson City, Tenn.	2,502	Jan.	1915	P. F. McDonald	
Ashtabula, Ohio	18,266	Jan.	1916		
Dayton, Ohio	116,577	Jan.	1914	Henry M. Waite	12,500
Sandusky, Ohio	19,989	Jan.	1916		
Springfield, Ohio	46,921	Jan.	1913	C. E. Ashburner	6,000
Westerville, Ohio	1,903		1915		
Alpena, Mich.	12,706	Apr.	1916		
Benton Harbor, Mich.	9,485		1916		
Big Rapids, Mich.	4,518		1914	W. J. Fairbairn	1,500
Cadillac, Mich.	8,375	Mar.	1914	O. E. Carr†	3,000
Grand Haven, Mich.	5,856		1914	E. M. Beck	2,500
Jackson, Mich.	31,433	Jan.	1915	G. C. Cummin	5,000
Manistee, Mich.	12,381	Apr.	1913	C. E. Ruger	2,000
Glencoe, Ill.	1,899		1914	H. H. Sherer	2,400
River Forest, Ill.	2,456		1914	Voted back to old form	
Winnetka, Ill.	3,168	Jan.	1915	R. L. Fitzgerald	2,400
Horicon, Wis.	1,881			E. C. Rehfield	1,000
Bloomfield, Iowa	2,028		1912	R. C. Bristow	680
Chariton, Iowa	3,794				
Clarinda, Iowa	3,832		1913	A. T. Wilson	1,700
Iowa Falls, Iowa	2,797		1914	E. L. Marriage	1,800
Webster City, Iowa	5,208		1915		
Morris, Minn.	1,685		1913	S. A. Siverts	1,600
Clark, S. D.	1,220		1912	J. E. Smith	960
Abilene, Kan.	4,118		1913	Kenyon Riddle	1,200
Mulberry, Kan.	997		1914	John Marion	
Amarillo, Tex.	9,957	Jan.	1914	M. H. Hardin	2,400
Denton, Tex.	4,732		1914	W. L. Foreman	
San Angelo, Tex.	10,321	Apr.	1916		
Sherman, Tex.	12,412		1915	K. M. Mitchell	
Taylor, Tex.	5,314		1914	Peter Schramm	
Terrell, Tex.	7,050		1913		
Tyler, Tex.	10,400	Apr.	1915		
Montrose, Colo.	3,252		1914	E. McDaniel	1,800
Collinsville, Okla.	1,324		1913	Claude Thorpe	2,000
Norman, Okla.	3,724				
La Grande, Ore.	4,843	Jan.	1914	F. J. Lafky	2,400
Snohomish, Wash.	3,244		1915	E. Colburn	
Roswell, N. M.	6,172		1914	W. M. Atkinson	1,800
Alhambra, Calif.	5,021	July	1915		
Bakersfield, Calif.	12,727	May	1915	Wallace M. Morgan	3,000
Glendale, Calif.	2,746			Thomas W. Watson	
Holtville, Calif.	729			D. A. Starbuck	
Inglewood, Calif.	1,536		1914	Abandoned	
San Diego, Calif.	39,578	May	1915	F. M. Lockwood	6,000
San Jose, Calif.	28,946	July	1916		
San Rafael, Calif.	5,934			W. A. Boland	
Santa Barbara, Calif.	11,659	Jan.	1917		
Phoenix, Ariz.	11,134	Jan.	1913	R. A. Craig	5,000
Tempe, Ariz.	1,473		1915		
Tucson, Ariz.	13,193		1915	C. K. Clark	
Maisonneuve, P. Q.					
Port Arthur, Ont.					

* Appointed since list was compiled.
† Resigned to go to Niagara Falls since list was compiled.

News of the Engineering World

Rogers Pass Holed Through

The five-mile long Rogers Pass tunnel of the Canadian Pacific Ry.—the longest tunnel on this continent—was holed through on Dec. 19. The alignment, grade and distance were perfect. The Rogers Pass tunnel was notable from three standpoints—its length, the novel method of tunneling adopted and the rapid rate of advance.

In order to expedite the work, two pioneer tunnels were driven parallel to the main tunnel, but about 50 ft. distant from it. From these drifts, crosscuts were driven to the centerline of the main tunnel, and from these the main headings were driven.

The object of this method was to make the most rapid advance possible, and this has been accomplished. Some of the monthly advances made, in the first half of 1915, are: East pioneer heading, 776 ft.; east main heading, 538 ft.; east enlargement for double track, 726 ft.; west pioneer heading, 932 ft.; west main heading, 752 ft.; west enlargement, 812 ft. On July 25 the two pioneer headings were stopped when they were only 5,202 ft. apart. From these points the main headings were driven forward, meeting on Dec. 19, the average advance per month for each main heading having been about 520 ft.

The contractors for the tunnel are Foley Bros., Welch & Stuart, of Winnipeg; A. C. Dennis is their superintendent. The work is under the direction of John G. Sullivan, Chief Engineer, Canadian Pacific Ry., and W. A. James, Engineer of Construction.

Coffer Blown Up To Complete Troy Lock and Dam

On Dec. 31, 1915, the remaining section of the earth coffer-dam in front of the old dam across the Hudson River at Troy, N. Y., was blown up as a step in the last stages of the new lock and dam construction at that point. The view herewith shows the coffer at the instant of the explosion. The coffer-dam was built upstream of the old dam so as to permit the removal of that structure.

The new Troy dam is a concrete structure about 1,400 ft. north of the old dam and about 1,000 ft. long. It holds back a head sufficient to provide a minimum of 14 ft. of water over the sills of the new lock, which is 45 ft. wide and 492.5 ft. long. The dam and lock are being built by the Federal Government and are designed to supplement the New York State Barge Canal, which empties into the Hudson just above Troy. The entire project comprises the construction of the new lock and dam, the removal of the old lock and dam and the dredging of a 12-ft. channel for about 30 mi. below the entrance of the Barge Canal. This should be complete by next season.

EXPLOSION CLEARS EARTH COFFER-DAM IN FRONT OF OLD TROY DAM

The Memphis Bridge Accident

The account of the Memphis falsework failure given last week (page 1291) is supplemented by the two accompanying views, taken Dec. 24, the day after the accident. One more bent was carried away the next day, Dec. 25. The weight of the bottom chords of the anchor arm helps to hold the remaining bents.

The washout was in the 418-ft. half-cantilever span at the west, or Arkansas, end of the bridge, between piers 3 and 4. The 186-ft. cantilever arm projecting from pier 3 (an extension from the fixed span 2-3) was being erected; the bottom chord had been placed. No part of the suspended span was erected. The falsework under the suspended span was washed out. There was no mattress on the river bed, and the bottom scoured out. No lives were lost, but a traveler was wrecked.

The last pin of the fixed span (span 2-3) was driven Dec. 22, after rush work to get the span together before the height of the flood wave. The portals also had been placed. On account of the high water the span has not yet been entirely swung; a little load is left on the falsework to help hold it in place. The river bottom here is protected by a mattress.

The river has been exceptionally high all season, but fell to nearly normal early in December. As the records show few rises at this period, no trouble was expected; the only high water at this time of year was in 1906 and 1911. But a rise commenced suddenly on Dec. 18, water coming in from the Ohio River, and the stage rose rapidly until at the time of the accident the water level had reached about 28½ ft. on the Memphis gage.

The only steel lost was some stringers for the highway floor, which were being used as traveler runways on the falsework.

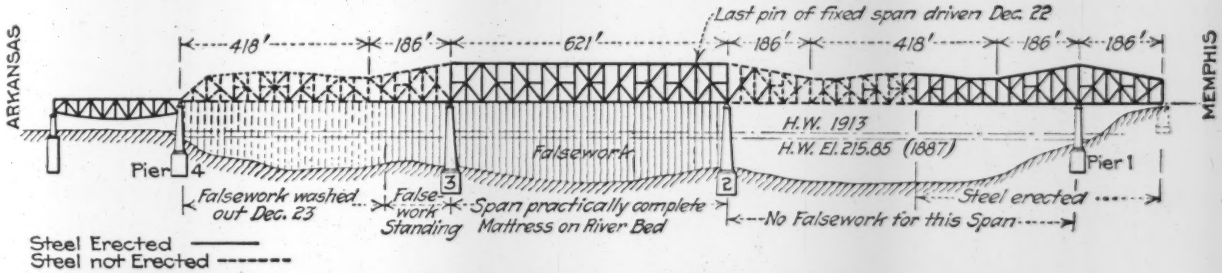
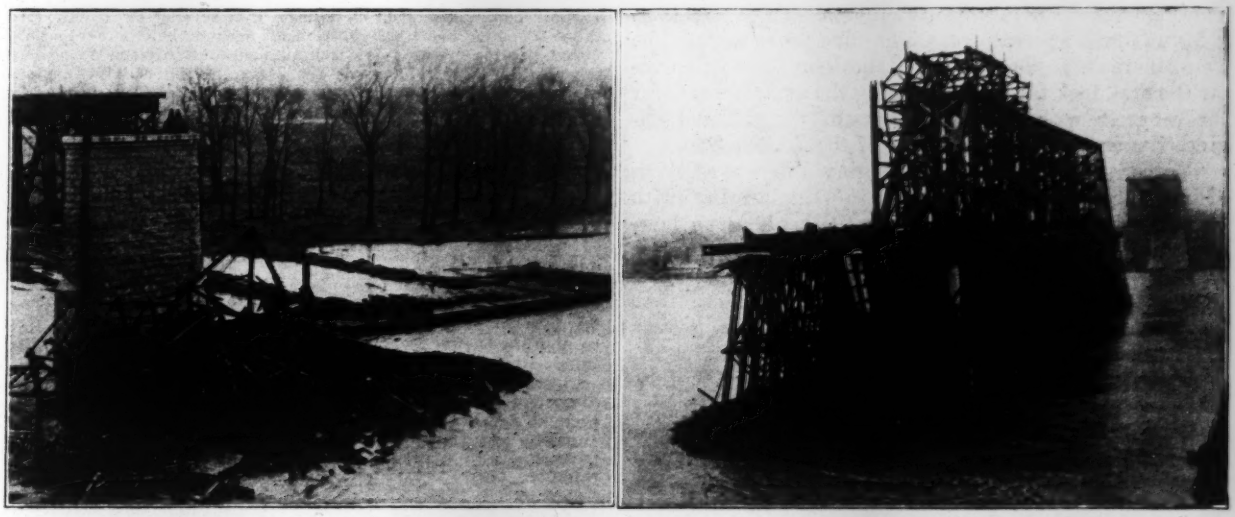


FIG. 1. DIAGRAM SHOWING ERECTION CONDITION OF NEW MEMPHIS BRIDGE

Flood Prevention at Dayton

With the first of the year the Miami Conservancy District takes permanent headquarters in a building all its own. The building has cost about $100,000 and is a gift to the district from E. A. Deeds, president of the Board of Directors of the district. It is of Bedford stone and concrete, with window and door frames of bronze. No wood enters into the construction at any point. For the present the second floor will be occupied by the offices of the directors, secretary and chief engineer, and by the files; the main floor by engineers and draftsmen; and the basement by a part of the engineering force and the appraisers, with rooms for a testing laboratory, for boring samples, for blueprints and photostat work, and by a dressing room with lockers for fieldmen.

With the formal organization of the district during the summer of 1915 the engineering work, which had been in the hands of the Morgan Engineering Co., of Memphis, Tenn., was taken over by the district, Arthur E. Morgan, president of the Morgan Engineering Co., continuing as Chief Engineer. A consulting board consisting

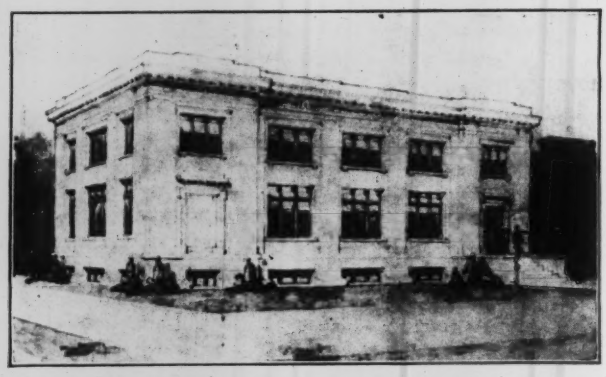


FIGS. 2 AND 3. SHORE END AND RIVER END OF OPENING 3-4 IN WHICH FALSEWORK WAS WASHED OUT

Fig. 2 shows pier 3, approach span at right. The half-cantilever between piers 3 and 4 will have its suspended span resting on pier 4. Fig. 3 shows pier 3 and the fixed span 2-3, with falsework and bottom chord of the cantilever arm in the foreground

of D. W. Mead and John W. Alvord, of Chicago, and Gen. H. M. Chittenden, of Seattle, Wash., has passed on the principal features of the plans. The announcement recently that General Chittenden was to give his entire time to the work was an error. Charles H. Paul, formerly construction engineer on the Arrowrock dam, recently joined the engineering force of the district as engineer on dam construction.

The dams for the retarding basins, exclusive of land, will cost between $7,000,000 and $8,000,000; the cost of channel improvement will be between $4,000,000 and $5,000,000; and the cost of railroad adjustments about $2,000,000. The entire cost of the project will be between $17,000,000 and $20,000,000. Mr. Morgan states that the "official plan" will be presented to the board in February. When this is completed, attention will be directed to issuing in the form of a report the results of the hydraulic and other investigations made during the preparation of the plan.



HEADQUARTERS BUILDING, MIAMI CONSERVANCY DISTRICT, DAYTON, OHIO

Engineering at Pan-American Scientific Congress

There was convened at Washington on Dec. 27 the second Pan-American Scientific Congress. (The first congress was held at Santiago, Chile, nine years ago.) The Pan-American Union through Hon. John Barrett, Director General, took the lead in making the arrangements for the congress with the coöperation of the national engineering and scientific societies of the United States.

The congress was divided into nine different sections, of which Section V was devoted to Engineering, with Gen. William H. Bixby as chairman. Other sections, however, dealt with matters of interest to engineers. Section VII, Mining, Metallurgy, Geology and Chemistry, had numerous papers on mining engineering; there were papers relating to irrigation and water-power in Section III. "Conservation of Resources" and important papers on engineering education were read in Section IV, Education.

Not only was the congress subdivided into nine separate sections, but each section was further divided into subsections, Engineering, for example, being subdivided into six subsections: (1) Civil, (2) Marine, (3) Electricity, (4) Reclamation, Sewerage and Water-Supply, (5) Mechanical, (6) Nomenclature, Standards, Mathematics, Surveys, Public Parks and Buildings.

The natural result of this minute subdivision was that most section meetings had very slim attendance. In fact the meetings of various scientific societies which were held in Washington at the same time—the American Economic Association, the American Sociological Society, the American Civic Association and a number of others—attracted much larger audiences than the Scientific Congress meetings.

The opening general sessions of the congress were made noteworthy by the participation of the Cabinet officers and other prominent Government officials and the members of the diplomatic staffs of South American countries. The Chilean ambassador, Eduardo Suarez, was president of the congress.

Over 75 papers were on the program for presentation in the Engineering Section alone. Most of them gave a general summary of engineering practice in the field covered by their subject. One paper, however, was of such timely interest and importance that it deserves special mention here. This paper, by Rome S. Brown, of Minneapolis, was entitled "Laws and Regulations Governing the Use of Water in Pan-American Countries." The author reviewed the origin and development of water laws in the United States and in Spanish-American countries and set forth clearly the legal obstacles which hinder the investment of capital in water-power development. The following is the concluding paragraph of his paper:

> There are millions of dollars of capital in the hands of American financiers ready for investment in water-power developments not only in the United States, but in all of the Pan-American republics, but which are withheld from such investments because of the financial obstacles presented in these various countries through an almost universal absence of a legislative policy which will allow such investments to be made with reasonable safety.

The Nitrate Industry

In a paper on the Chilean nitrate industry, by Enrique Cuevas, of Chile, the following figures were given, which are of interest in connection with projects for manufacturing nitrates in the United States by hydro-electric power:

> According to the latest official report of the inspector general of the nitrate deposits, the zone of nitrate-bearing ground comprises 200,000 square kilometers, of which less than 3% has been surveyed and prospected. In this surveyed portion alone there remains enough nitrate to supply the world for an additional 100 years, measured at the present rate of production. These calculations have been made on an extremely conservative basis and take no account whatever of known areas of low-grade material.
>
> The industry is divided into two principal operations: (1) Mining and transportation of the crude material, and (2) production of nitrate. Methods employed in both branches leave room for improvement. Under present practice, considering average value of raw material, wages, etc., the cost of producing commercial nitrate is estimated at 50c. per quintal (101.44 lb.)
>
> Trade specifications require a content of nitrate of sodium equal to at least 95%. This is used as a fertilizer. The refined product, with over 96% nitrate of sodium, is applied to manufacturing uses, such as glass, explosives, fusing mixtures, nitric acid, etc.
>
> There can be no competition between Chilean nitrate and atmospheric nitrate under any conditions which may be foreseen at present. The deposits of the great Pampa Salitrera (nitrate plain) are too vast and the business of manufacturing and marketing the material is too well systematized.

Papers discussing the achievements and possibilities in the artificial production of nitrates were read by Prof. G. A. Roush, of Lehigh University, and Thomas H. Norton, of the Department of Commerce.

The number of delagates from South and Central America present at the congress was about 200. Those registered from the United States numbered about 1,000. A large proportion of them came to Washington chiefly to attend the other scientific societies which held sessions at the same time as the congress. The main purpose of the congress, however, the promotion of cordial relations with leading men of the other American republics, was successfully accomplished.

Congress has appropriated $70,000 for the expenses of the congress and the entertainment of the foreign delegates. After the close of the congress on Jan. 8, these visitors are to be taken on an excursion, visiting Baltimore, Philadelphia, New York, Boston, Niagara Falls and Pittsburgh.

University Solves Ohio Flood Problem in New Way

The Ohio State University *News* for Dec. 17, 1915, has the following publicity notice:

The conservancy engineers who have been studying flood protection on the Scioto and Olentangy Rivers have abandoned the plan of securing protection by merely widening the channels, as was at first proposed, which would secure restricted benefits only locally at places along the streams where such widening would be done. They now contemplate a project which will secure more widespread benefits.

In this connection a novel plan is proposed in a bulletin just issued by the Ohio State University. This plan proposes to turn Scioto flood waters into Lake Erie. The surprising part of this project is that it proposes to give relief to the Sandusky Valley in this way. Also the area west of Marion and the Scioto and Olentangy valleys are protected. Army engineers have, however, tentatively approved the general plan as being able to accomplish these seemingly conflicting ends. Not only flood protection, but many other benefits, such as water-supplies for all cities in both valleys, water power, a navigable route 35 mi. long with lakes, are to be secured without added expense, it is claimed.

Since perpetual stream regulation has now been entered upon at several places in the state by conservancy districts under the Vonderheide law, the question of what to do with our streams in the 50 or 100 yr. between great floods is now very pertinent.

The bulletin referred to discusses, in the first seven chapters, the above questions from a state-wide standpoint, and then proceeds with a specific application to the treatment of the Sandusky and Scioto Rivers. The remarkable conclusion is reached that it will be cheaper and more beneficial to protect both the Sandusky and Scioto valleys rather than just one.

The bulletin is No. 15 of the College of Engineering. Five hundred copies have been ordered by the city engineer of Columbus for local distribution. A limited edition is for free distribution by the university.

The Cleveland & Youngstown R.R. freight terminal at Cleveland, Ohio, authorized by the voters on Nov. 3, 1915, is to be started soon. Bids are to be taken for the initial structure some time this month.

An Interesting Lawsuit has been begun in New York City, involving the status of the only woman member of the American Society of Civil Engineers. Nora Stanton Blatch, listed in the 1915 membership list of the society as an assistant engineer, Public Service Commission, First District, New York, has made an application for a writ of mandamus requiring the society to admit her to associate membership. She is a graduate in civil engineering of Cornell University and has been a junior member of the society since 1906. Having become 32 yr. of age, she has been dropped from the membership rolls in accordance with the constitution of the society, which prohibits membership in this grade of persons over 32 yr. old.

A National Conference on Immigration and Americanization is to meet in Philadelphia on Jan. 19 and 20. The object of the conference is to discuss means of systematically inducing alien workmen to take out naturalization papers and become citizens of the United States. The passage of laws concerning the employment of alien labor on public works makes this conference of especial interest to engineers and contractors. Among the members of the general committee in charge of the conference are Messrs. R. T. Crane, Coleman du Pont, Thomas A. Edison, Howard Elliott and Samuel Rea. Frank Trumbull, 18 West 34th St., New York City, is chairman of the committee.

Stray Current Worked a Signal Relay on the Nashville, Chattanooga & St. Louis Ry., Dec. 23, 1915, causing a wreck between a fast freight and a passenger train at a point 91 mi. south of Nashville. An engineer, a fireman and 13 negroes were killed. The Interstate Commerce Commission sent two signal engineers and two inspectors to investigate the accident. The railway cleared the track and had the injured under care within 30 min. after the collision. The statement of the railroad is as follows:

The train movement in the territory in which the wreck occurred is made under the controlled manual-block system. Signals cannot be given to opposing trains without cooperation of the operators at each end of the block. Investigation shows that the signal to the south-bound freight train was not given by the operator but was caused by current from some outside source picking up the signal relay. The signal wires of the railroad company and the Western Union wires were on the same pole, the signal wires being underneath those of the Western Union. It is reported that Western Union forces were working on their wires in that vicinity Thursday afternoon.

Federal Appropriations for Public Works recommended to Congress by the executive departments for the fiscal year ending June 30, 1917, contain a number of items of interest to engineers. The Treasury Department asks for $10,054,000 for the purchase of sites and construction of public buildings and $450,000 for the purchase of mechanical equipment for existing buildings. These estimates cover work to be done on projects previously authorized by Congress. The War Department asks for $25,791,972 in connection with the fortifications and defenses of the United States and $45,844,458 for carrying on work on existing river and harbor projects. The Panama Canal estimate is $19,301,369, including new work in addition to operation and maintenance, with a further sum of $8,234,100 for fortifications. The Interior Department's principal items are $8,938,000 for the Reclamation Service and $1,000,000 for irrigation on Indian reservations. The Department of Commerce calls for $2,252,300, which includes public work for the Lighthouse Service and for the Bureau of Standards. The estimate for public works under the Navy Department is $3,959,000, not including any appropriation for construction of vessels.

Typhoid Fever in the United States Army in 1914 reached a total of only seven cases and three deaths among a mean strength of 98,649 officers and enlisted men stationed in the United States and elsewhere, according to the annual report of the surgeon-general, United States Army. Only two of the seven cases had received the complete course of vaccine immunization and neither of these died. Voluntary immunization was begun in 1909, but the treatment was not made compulsory for the entire army until 1911. The army typhoid case and death rate per 1,000 from 1905 to 1914 has been as follows:

Year	Case Rate	Death Rate	Year	Case Rate	Death Rate
1905*	3.57	0.30	1910	2.37	0.17
1906*	5.66	0.28	1911	0.76	0.10
1907*	3.53	0.19	1912	0.29	0.05
1908	3.02	0.26	1913	0.05	0.00
1909	2.96	0.28	1914	0.11	0.05

*Officers not included.

Among troops scattered along the American border and in the large camps in Texas there has not been a case of typhoid fever since June 4, 1912.

Promotion in the United States Navy by merit instead of by seniority is recommended in the annual report of Secretary Daniels. The objections to the long-established system of promotion by seniority have been accentuated by the action of the last Congress in abolishing the system of compulsory retirement of officers which has been in force since 1899. This "plucking" system, so called, relieved to some extent the stagnation in the lower grades and the very short term of service in the higher grades, but involved so many other serious objections that its repeal in Congress was by almost unanimous vote. If the efficiency of the navy is to be maintained, something different from the old seniority system of promotion must be established. Secretary Daniels recommends that in order to avoid promotion by political influence the recommendations for promotion should be made by the naval officers themselves. He says: "A system of promotion based practically upon the vote of the officers senior to the grade under consideration would unquestionably select approximately the most efficient officer in that grade."

PERSONALS

Mr. Walter H. Kestin has been appointed Sales Manager of the Concrete Machinery Department of the Northwestern Steel and Iron Works, of Eau Claire, Wis.

Mr. T. L. Hughes has been appointed District Sales Manager, at New Castle, Penn., of the Lehigh Portland Cement Co. He succeeds Mr. P. A. Jandernal, who recently resigned.

Mr. George J. Adamson, formerly Special Engineer of Construction of the Union Pacific R.R. at Omaha, Neb., has been made Division Engineer of the Wyoming division of the road, with headquarters at Cheyenne, Wyo.

Mr. Trygve D. Yensen, First Assistant in the engineering experiment station, Department of Electrical Engineering, University of Illinois, has been promoted to be Research Assistant Professor of Electrical Engineering.

Mr. George F. Maitland, M. Am. Soc. C. E., Division Engineer, Maintenance-of-Way, of the Union Pacific R.R., at Cheyenne, Wyo., has been made Division Engineer of the Kansas Division, with headquarters at Kansas City, Mo.

Mr. John B. Dickson, General Superintendent of the Erie R.R., at Cleveland, has been made General Manager. He succeeds Mr. R. S. Parson, who has been promoted to be Assistant to the President and Chief Engineer with headquarters in New York, as noted elsewhere.

Mr. Arthur E. Sweet, of Topeka, Kan., General Manager of the Chicago, Rock Island & Pacific Ry., has been elected Vice-President and General Manager in charge of operation of the Denver & Rio Grande R.R. He will succeed Mr. F. L. Brown, who recently resigned.

Mr. Oscar Erlandsen, M. Am. Soc. C. E., President of the Metropolis Engineering Co., of Jamaica, Long Island, has been elected Chairman of the Borough Planning Committee by the Board of Directors of the Chamber of Commerce of the Borough of Queens, New York City.

Mr. Frederick A. Benz, formerly Assistant Engineer of the Buffalo, Rochester & Pittsburgh Ry., has been promoted to the rank of Division Engineer of Divisions 1 and 2 and the Erie division, with headquarters in Rochester. He has been in the service of the road since 1906.

Mr. O. S. Beyer, Jr., Jun. Am. Soc. M. E., General Foreman of the Chicago, Rock Island & Pacific Ry. at Horton, Kan., has been appointed First Assistant in the engineering experiment station in the Department of Railway Engineering of the University of Illinois, Urbana, Ill.

Mr. J. H. Fishback, for a number of years Assistant to the Secretary of the Interstate Commerce Commission, has resigned to enter upon the practice of interstate-commerce law. He has been chairman of the important board of reference of the commission and also a member of the Board of Suspensions.

Mr. W. T. Lechlider, Superintendent of the Cleveland division of the Baltimore & Ohio R.R., has resigned to accept an appointment as Vice-President and General Manager of the River Terminal Railway Co., which operates the railroad system at the Corrigan & McKinney Co.'s plant in the upper Cuyahoga Valley in Ohio.

Mr. Virgil G. Marani, M. Am. Soc. C. E., of Marani & Moore, Consulting Engineers, 1900 Euclid Building, Cleveland, Ohio, has opened offices at 205 Monroe St., Chicago, as Consulting Engineer for the United States Gypsum Co. He will retain his interest in consulting work in Cleveland. Mr. Marani was formerly Building Inspector of the City of Cleveland.

Mr. Alfred H. Hartman, M. Am. Soc. C. E., Division Engineer of the Baltimore Sewerage Commission, will carry to completion the work of the combined storm-water, low-level and high-level sanitary divisions. Mr. Hartman has been connected with the Baltimore work since 1906, serving first as Designing Engineer and for the past six years as Division Engineer.

Messrs. Virgil D. Allen, Building Commissioner of the City of Cleveland, Ohio, for the past four years, and Irwin S. Osborn, Assoc. M. Am. Soc. C. E., Consulting Engineer, Toronto, Can., have formed the Allen-Osborn Co., Consulting Engineers, with offices at 955 Rose Building, Cleveland, Ohio. The company will specialize in factory and mill-building construction, municipal problems and garbage reduction.

Messrs. John Cotton Dana, Librarian, and Harland Bartholomew, Secretary, have tendered their resignations to the City Plan Commission of Newark, N. J. Mr. Dana, it is stated, gave as his reason the fact that he had a great many other responsibilities and that his health would not permit him to continue his work as librarian. Mr. Bartholomew is said to have been offered a similar position with a considerably larger salary by another city.

Mr. E. L. Dunbar has resigned as Director of the Water-Works Department of Bay City, Mich., after 44 years' service. In view of his long and meritorious service, the Board of Aldermen of that city has passed the following resolution:

Resolved, That we do hereby voice our appreciation and sincere thanks for the many years of faithful and untiring efforts for the advancement and improvement of the department; that we regret to lose his valuable services and do hereby extend our hearty wishes that he will enjoy life.

Mayor Louis P. Fuhrmann, of Buffalo, N. Y., which went under the commission-plan of city government on Jan. 1, has announced the appointments of the bureau heads. Only two new appointments are made: Mr. William F. Schwartz, president of a street-sweeping machine company, has been appointed Street Commissioner; and Mr. Thomas W. Kennedy has been appointed Water Commissioner. Mr. George H. Norton has been reappointed Deputy Commissioner of Public Works and City Engineer, and his salary has been increased from $4,000 to $5,000 per annum.

Messrs. J. F. Poland and R. E. Miller, Jun. Am. Soc. M. E., have formed a partnership under the name of the Poland-Miller Co., with offices in the Erie Building, Cleveland, Ohio. Mr. Poland was formerly associated with the Brown Hoisting Machinery Co. as a civil and structural engineer, and in the same capacity with the J. W. Frazier Co. Mr. Miller has recently been associated with the J. W. Frazier Co. as a mechanical and electrical engineer. The firm has been organized to conduct a general engineering business, specializing in the design and inspection of coal, coke and ore handling machinery, docks and industrial plants.

Lieut.-Col. Edward Jadwin, Corps of Engineers, U. S. A., has been transferred to Pittsburgh, where he will have charge of the River and Harbor District of Pittsburgh and of the improvement work in connection with the Monongahela, Alleghany and upper Ohio rivers. Colonel Jadwin has been attached to the office of the Chief of Engineers at Washington. He graduated from West Point in 1890 and has had various details, including that of Division Engineer on the Panama Canal construction work. He will be succeeded in Washington by Lieut.-Col. Charles Keller, Corps of Engineers, M. Am. Soc. C. E., who is now in charge of the River and Harbor work at Mobile, Ala.

Mr. Henry W. Hodge, M. Am. Soc. C. E., Consulting Engineer, New York City, has been appointed by Governor Whitman a member of the Public Service Commission of New York City, to succeed Commissioner Robert C. Wood, who resigned his office on Jan. 4. Mr. Hodge is one of the leading men in the bridge-engineering field, having for many years been a partner of the late Alfred P. Boller, of the firm of Boller & Hodge. He is a director of the American Society of Civil Engineers. The appointment by Governor Whitman of an engineer of ability and reputation as a member of the Public Service Commission is in accordance with the recommendations heretofore urged by the engineering societies and will be heartily commended by the profession.

Mr. Robert S. Parsons, M. Am. Soc. C. E., General Manager of the Erie R.R. at Chicago, has been appointed Assistant to the President and Chief Engineer, with headquarters at New York City. He was born in Hohokus, N. J., in 1873, and graduated from Rutgers College as a bachelor of science in 1895. He received the degree of civil engineer from Rutgers in 1900. In 1895 Mr. Parsons entered the employ of the Erie as transitman on the New York division, and since then has been consecutively Assistant Engineer of the Buffalo division, Assistant Engineer of the New York, Susquehanna & Western and the Wilkes-Barre & Eastern railroads; Division Engineer of the same roads and Engineer of Maintenance-of-Way of the Erie at Cleveland; Superintendent of the Susquehanna division of the Erie; Superintendent of the New York division of the same road and New Jersey & New York R.R., Greenwood Lake R.R. and Northern Railroad of New Jersey. In 1913 he was made Assistant General Manager of the Erie at Chicago, and later became General Manager, which position he has held up to the present time.

OBITUARY

John Andrews, one of the pioneer contractors and builders of Kansas City, Mo., died at his daughter's home in that city, Dec. 28, at 65 years of age.

Thomas F. Richardson, M. Am. Soc. C. E., former Chief Civil Engineer of J. G. White & Co., New York City, died at

his home in Rutherford, N. J., Dec. 26, 1915, aged 60 years. A biographical sketch will appear in a later issue.

Gen. Grenville M. Dodge, Chief Engineer of the Union Pacific R.R. during its construction across the Western plains, died, Jan. 3, at his home in Council Bluffs, Iowa, in his 85th year. A biographical sketch and portrait will appear in a later issue.

Calvin G. Goodrich, President of the Twin City Rapid Transit Co. and St. Paul City Railway Co., died at his home in Minneapolis, Dec. 28. He was born in Oxford, Ohio, in 1856 and educated in the schools of Minneapolis. For a short time he was engaged in the wheat commission business and in 1877 entered the service of the Minneapolis Street Railway Co. as a bookkeeper. He became successively Superintendent, General Manager and Vice-President, and in 1909 was elected President. Since 1900 he had also been President of the Duluth-Superior Traction Co., owned by the same interests.

ENGINEERING SOCIETIES

COMING MEETINGS

ALABAMA ASSOCIATION OF HIGHWAY ENGINEERS.
Jan. 8. Annual meeting in Montgomery. Secy., J. B. Converse, City Engineer, Selma, Ala.

LOUSIANA ENGINEERING SOCIETY.
Jan. 8. Annual meeting in New Orleans. Secy., W. T. Hogg, State Museum Bldg., New Orleans, La.

WESTERN SOCIETY OF ENGINEERS.
Jan. 12. Annual meeting in Chicago. Acting Secy., E. N. Layfield, 1735 Monadnock Block, Chicago, Ill.

ENGINEERS' SOCIETY OF PENNSYLVANIA.
Jan. 14. Annual meeting in Harrisburg. Secy., E. R. Dasher, 30 So. Front St., Harrisburg, Penn.

CENTRAL RAILWAY CLUB.
Jan. 14. Annual meeeting in Buffalo, N. Y. Secy., H. D. Vought, 95 Liberty St., New York City.

WESTERN PAVING BRICK MANUFACTURERS' ASSOCIATION.
Jan. 14-15. Annual meeting in Kansas City, Mo. Secy., G. W. Thurston, 416 Dwight Bldg., Kansas City, Mo.

MONTANA INSTITUTE OF MUNICIPAL ENGINEERS.
Jan. 17-19. Annual meeting at Billings, Mont. Secy., C. C. Widener, Bozeman, Mont.

ENGINEERS' SOCIETY OF WESTERN PENNSYLVANIA.
Jan. 18. Annual meeting in Pittsburgh. Secy., E. K. Hiles, 2511 Oliver Building, Pittsburgh, Penn.

KANSAS ENGINEERING SOCIETY.
Jan. 18-19. Annual meeting at Kansas State Agricultural College. Secy., C. A. Foster, Topeka, Kan.

AMERICAN WOOD-PRESERVERS' ASSOCIATION.
Jan. 18-20. Annual convention in Chicago. Secy., F. J. Angier, Mount Royal Station, Baltimore, Md.

THE FRANKLIN INSTITUTE.
Jan. 19. Annual meeting in Philadelphia. Secy., R. B. Owens, Philadelphia, Penn.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
Jan. 19-20. Annual meeting in New York City. Secy., C. W. Hunt.

NATIONAL DRAINAGE CONGRESS.
Jan. 19-21. Annual meeting, Cairo, Ill. Vice-Pres., Edmund T. Perkins, First National Bank Building, Chicago, Ill.

NATIONAL SOCIETY FOR THE PROMOTION OF INDUSTRIAL EDUCATION.
Jan. 20-22. Annual convention in Minneapolis. Secy., Alvin E. Dodd, 140 West 42nd St., New York City.

ILLINOIS CHAPTER: AMERICAN WATER-WORKS ASSOCIATION.
Jan. 25-27. Annual meeting, University of Illinois, Urbana. Secy., Dr. Edward Bartow, Urbana, Ill.

ILLINOIS SOCIETY OF ENGINEERS AND SURVEYORS.
Jan. 26-28. Annual meeting, University of Illinois, Urbana, Ill. Secy., E. E. R. Tratman, Wheaton, Ill.

WISCONSIN HIGHWAY COMMISSIONERS' ASSOCIATION.
Jan. 31. Annual meeting in Madison. Secy., G. H. Mainwaring, Gotham, Wis.

INDIANA SANITARY AND WATER-SUPPLY ASSOCIATION.
Feb. 2-5. Annual meeting in Indianapolis. Asst. Secy., W. F. King, Indianapolis, Ind.

INDIANA ENGINEERING SOCIETY.
Feb. 3-5. Annual meeting in Indianapolis. Secy., Charles Brossmann, 1616 Merchants Bank Bldg., Indianapolis, Ind.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Feb. 4. Mid-winter meeting in Chicago. Secy., E. B. Burritt, 8 W. 40th St., New York City.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
Feb. 8-9. Mid-winter meeting in New York City. Secy., F. L. Hutchinson, 29 W. 39th St., New York City.

OHIO ENGINEERING SOCIETY.
Feb. 9-11. Annual meeting in Columbus. Secy., John Laylin, Norwalk, Ohio.

NINTH CHICAGO CEMENT SHOW.
Feb. 12-19. First Infantry Armory and Coliseum. Secy., R. F. Hall, 208 South La Salle St., Chicago, Ill.

CONNECTICUT SOCIETY OF CIVIL ENGINEERS.
Feb. 14-15. Annual meeting at New Haven. Secy., J. Frederick Jackson, Chamber of Commerce Bldg., New Haven, Conn.

AMERICAN CONCRETE INSTITUTE.
Feb. 14-17. Annual convention in Chicago. Secy., John M. Goodell, 1418 Walnut St., Philadelphia, Penn.

AMERICAN INSTITUTE OF MINING ENGINEERS.
Feb. 14-17. Annual meeting in New York City. Secy., Bradley Stoughton, 29 W. 39th St., New York City.

ASSOCIATION OF ONTARIO LAND SURVEYORS.
Feb. 15. Annual meeting in Toronto, Can. Secy., L. V. Rorke, Parliament Building, Toronto, Ont.

SECOND NATIONAL CONFERENCE ON CONCRETE ROAD BUILDING.
Feb. 15-18. At Chicago. Secy., J. P. Beck, 208 South La Salle St., Chicago, Ill.

IOWA ENGINEERING SOCIETY.
Feb. 16-18. Annual meeting at Des Moines. Secy., J. H. Dunlap, Iowa City, Iowa.

NEW ENGLAND ASSOCIATION OF GAS ENGINEERS.
Feb. 16. Annual meeting in Boston. Secy., N. W. Gifford, 38 Central Sq., East Boston, Mass.

AMERICAN CONCRETE PIPE ASSOCIATION.
Feb. 17-18. Annual convention in Chicago. Secy., E. S. Hanson, 538 South Clark St., Chicago, Ill.

AMERICAN ROAD BUILDERS' ASSOCIATION.
Feb. 22-25. Annual convention in Pittsburgh. Secy., E. L. Powers, 150 Nassau St., New York City.

The New England Water-Works Association holds its annual meeting on Jan. 12 in Boston at the Hotel Brunswick.

The Michigan Engineering Society will hold its 37th annual convention at Grand Rapids Jan. 18 to 20. The secretary is S. J. Hoexter, Ann Arbor, Mich.

The Arkansas Association of Surveyors and Civil Engineers held its seventh annual convention in Little Rock, Dec. 30 to 31. The secretary is J. O. Jones, of Dardanelle.

National Electric Light Association—The 39th convention will be held during the week beginning May 22, 1916, in Chicago. The headquarters will be at the Congress and Auditorium hotels, where it is proposed all meetings shall be held as well as the annual exhibit. The secretary is T. C. Martin, 29 W. 39th St., New York City.

The Society of Automobile Engineers holds its annual banquet at Hotel Plaza, New York City, on Jan. 6. The society will be addressed on that occasion by Josephus Daniels and Major-General Leonard Wood, U. S. A., on "Army Transport Problems." On Jan. 5 the Standards Committee reports, while on Jan. 6 there will be addresses and papers, both in the morning and in the afternoon.

The Alabama Association of Highway Engineers will hold its annual meeting at Montgomery on Jan. 8. Subjects to be discussed are: "Should Engineers be Licensed to Practice Their Profession"; "What Relation Has County Engineer to Board of Revenue or to County Commissioners"; "Should Counties Pay Their Highway Engineers a Specified Salary or Should They Be Paid for Engineering Service on a Percentage Basis for Work Done?"

American Civic Association—The eleventh annual convention of the association was held in Washington, D. C., in conjunction with the second Pan-American Scientific Congress, Dec. 28 to 31. The program was divided into nine sections, which included City Planning, Better Industrial Housing, Planning for a Small Town, and Country Planning. The offices of the American Civic Association are at 914 Union Trust Building, Washington, D. C.

American Civic Association—The annual meeting was held at Washington, D. C., Dec. 28 to 31, and was largely devoted to city and village planning. A session was taken up with the development of the national parks and another with the use of schools as community centers. In the election of officers, Miss Margaret Wilson, daughter of the President, and Hon. Geo. B. McAneny, of New York, were chosen Vice-Presidents. J. Horace McFarland, of Harrisburg, Penn., was re-elected President.

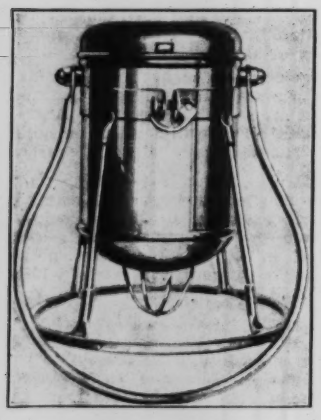
The Associated Technical Men, an organization devoted to the professional interests of technical men, will hold a meeting and dinner at the Morrison Hotel, Chicago, on Jan. 18. A. J. Hammond, principal assistant engineer, Chicago Union Station Co., will speak on "Engineers as Expert Witnesses." A. E. Gibson is general secretary, with offices at 1215 Monadnock Block, Chicago. A. M. Wolf is editor of the "Technician," a monthly bulletin the purpose of which is to distribute information "regarding the present unsatisfactory professional status of the technical profession, and means by which it can gain the proper recognition of the other learned professions."

American Society of Agricultural Engineers—The ninth annual meeting was held at the Sherman Hotel, Chicago, Dec. 28-30. One session was devoted to modern farm conveniences, such as water supply, sewerage, lighting and heating; this included a talk by Burton J. Ashley on small sewage disposal plants. Another session was devoted to farm tractors, their design and their economic aspects. One speaker considered them as substitutes for horses, and thought the weight should not exceed 3,000 lb. for a tractor that can be used for general purposes. Other speakers thought the tractor could not economically replace the horse for all the odds and ends of farm work. In a talk on the problems of agricultural engineering research, Prof. P. S. Rose (University of Wisconsin) advocated an investigation of power requirements and the proper combination of units for various purposes. At present dealers in machinery are not able to make such combinations and the result is that there are many "misfit outfits." The matter has been taken up by the gas engine manufacturers, who have already issued numerous data sheets giving extremely useful information. Prof. Rose proposed to have such work done by the coöperation of manufacturers, the laboratories of engineering colleges, and the Department of Agriculture (through its Bureau of Rural Engineering).

Appliances and Materials

Shadowless Safety Lantern

The accompanying view shows a new electric safety lantern designed by a practical railroad man and now being made by the Federal Sign System, Lake and Desplaines St., Chicago, Ill. Danger of explosion in working around gas, oil and other inflammables is prevented by a double set of strong guards inclosing the incandescent bulb. Shadows are prevented by placing the lamp beneath the battery compartment. The outer guard forms the standards on which the lantern rests when set down.



SHADOWLESS SAFETY LANTERN

* * *

New Safety Tread

A new form of safety tread for stairs, etc., has been devised by the American Mason Safety Tread Co., of Lowell, Mass. This product carries the trade name "Black Diamond." The body of the tread plate is made of asphaltum and carborundum. On top of this is a steel or brass skeleton frame with large diamond-shaped openings. The effect of traffic is to force the somewhat plastic material up through the grid, so that the tread wears down smooth. The metal frame confines the edges, and the cross-bars serve to drain the plate.

* * *

Black-Line Reproduction of Blueprints

The process of reproducing drawings by inked impressions from an etched plate of gelatin or paste has been perfected by the Rexim Co., 125 S. 11th St., Philadelphia, which has placed on the market sets of operating apparatus.

The following items comprise a typical outfit: A table 50x76 in. with a ground-glass top, a double boiler for heating the transfer paste, an 80-in. gutter for the table, two ink rollers, 2 lb. of "Eureka" process ink, an ink knife, a paste knife, a pan, a strainer, a large spoon, two ink plates, a thermometer, a pad, 50 lb. of "Eureka" process ink, an ink knife, a paste knife, a pan, a strainer, a large spoon, two ink plates, a thermometer, a pad, 50 lb. of "Eureka" paste, 1 lb. of "Eurepa" process varnish, a sponge, 1 lb. of "Eureka" powder, a pitcher, a dipper, a zinc-top stand and a hydrometer.

The paste is melted in the boiler and spread out on the glass-top table. When the surface is right for receiving it, an exposed blueprint (not washed) is laid face down on the paste, left for a few seconds and removed. The unexposed lines of the drawing on the blueprint leave slight etchings on the paste surface, which are inked with a roller, as in printing from any engraving. Prints are made in the usual way, by pressing paper upon the inked paste plate. The ink used will transfer to tracing cloth as well as to paper and may be had in different colors. By reinking the plate any reasonable number of reproductions may be taken. The paste is remelted and used over and over again. There are certain precautions to be observed, and a certain amount of skill is required, but the process itself is simple.

* * *

Graphic-Record Supplies

A large line of devices for the graphic representation of facts has been prepared and is being marketed by the Educational Exhibition Co., 26 Custom House St., Providence, R. I. There are authentic maps of the United States (1, 3 and 6 ft. wide), of the several states, counties, important cities and railway lines. For map marking there are colored-head needle-pointed pins; ring, number, flag, markable and other map pins; gummed paper letters and symbols, etc. Among the charting papers are various simple- and logarithmic-scale sheets, and curve cards in file sizes.

* * *

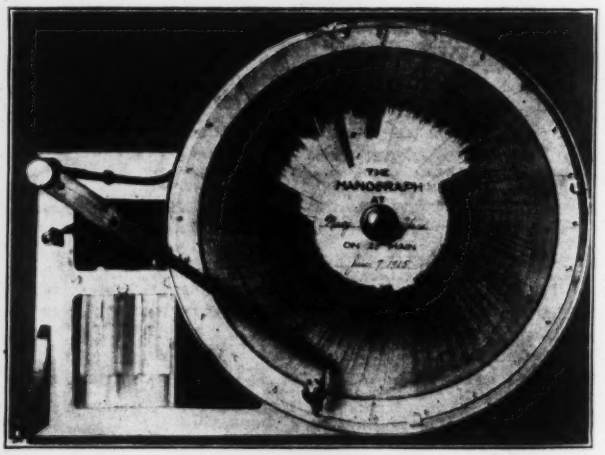
Uniform-Pressure Street Flusher

The Studebaker Co., South Bend, Ind., has placed on the market a 750-gal. street flusher which can be used for a variety of purposes. The horizontal cylindrical tank is of sheet steel, electrically welded. The chief feature of the flusher is a single-stage centrifugal pump which discharges at a pressure of 40 lb. per sq.in. The pump is driven by a four-cylinder four-cycle automobile gasoline engine at the rate of 800 r.p.m. The outfit may be used for pavement flushing, spraying trees and shrubbery, or for applying cold asphaltic or tar oils in roadbuilding. In an emergency it can be used to throw one or two streams for firefighting.

* * *

Recording Differential Pressure Gage

A new sensitive recording differential pressure gage, called the "Manograph," intended particularly for use with pitot tubes in studying leaks in water-works systems, has been devised by Paul Lanham (pitometer operator in the office of the Engineer Commissioner of the District of Columbia) and is being made and sold by him at Lanham, Md. A closed manometer U-tube with mercury is used, one leg connected to the "upstream" orifice of the pitot tube and the other to the "down-stream" side. A vertical rod, bearing a sharp platinum needle, passes through a stuffing-box and into the high-pressure compartment of the manometer. This rod is reciprocated in and out of contact with the mercury by a water motor or some equivalent. A pen arm, moving across a chart, is attached to the movable rod. The pen is not normally in contact with the chart and makes no record until an electric circuit is completed that includes an electromagnet which forces the pen against the paper. This circuit is completed when the platinum needle touches the shifting mercury level. The pen then draws a series of regular lines across the chart as shown in the view, the beginning of these lines plotting the change in differential pressure and accordingly the flow. To keep the pen from drawing on the return stroke of the contact rod, an auxiliary contact is opened by the motor. None of the moving parts derive their energy from the pressure being measured, so that the sensitiveness does not depend on the recording mechanism.



LANHAM'S DIFFERENTIAL PRESSURE RECORDER